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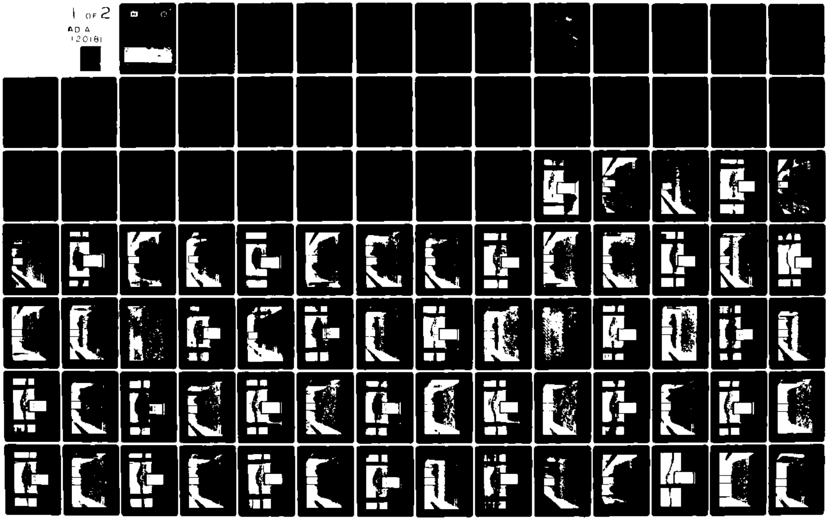

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TECHNICAL REPORT HL-82-14

KAHULUI BREAKWATER STABILITY STUDY
KAHULUI, MAUI, HAWAII

Hydraulic Model Investigation

by

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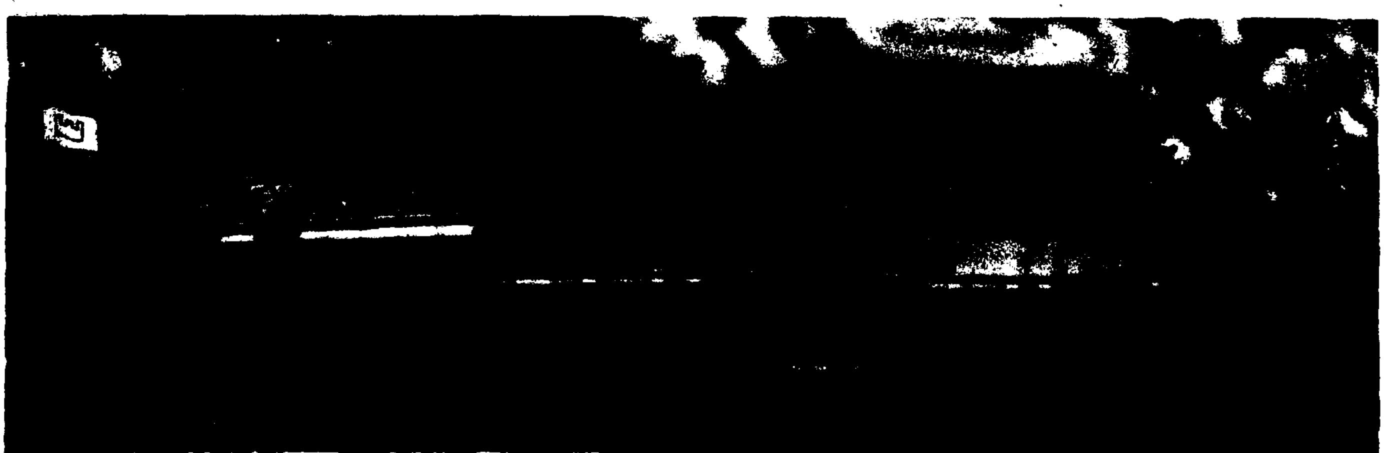
July 1982
Final Report



Prepared for U. S. Army Engineer Division, Pacific Ocean
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A hydraulic model investigation was conducted at geometrically undistorted, linear scales of 1:33, 1:36, and 1:40, model to prototype, to evaluate the stability against wave attack of proposed rehabilitation designs for two areas, each on the harbor sides of the east and west breakwaters at Kahului Harbor, Maui, Hawaii. A proposed rehabilitation design for the sea-side slope of the west breakwater at sta 18+50 and the existing sea-side slope protection on the west breakwater at sta 21+25 also were evaluated for stability against
(Continued)

20. ABSTRACT (Continued)

wave attack. Where the proposed designs failed, additional tests of alternative plans were conducted until stable design sections were found. All plans were tested for the worst breaking wave conditions that could be produced for the selected wave periods, water depths, and bathymetry seaward of the test sections.

The existing 19-ton tribars on the sea-side slope of the west breakwater at sta 21+25 proved to be stable, and six plans (three dolos and three tribar) were found acceptable for the proposed harbor-side slope rehabilitation. With the addition of a concrete rib cap on the crown of the west breakwater at sta 18+50, 11-ton and 5-ton tribars were found to be stable on the sea- and harbor-side slopes, respectively. For the east breakwater at sta 26+10, 9-ton tribars showed very good stability when placed on a 1V-on-2H harbor-side slope. A concrete rib cap was added to stabilize the crown and upper sea-side slope of the east breakwater at sta 23+35 and 9-ton tribars placed on a 1V-on-2H slope provided stable protection for the harbor-side slope.

The stabilities of all plans found acceptable are dependent upon trenching and/or special placements of the toe armor units, as described for each alternative design. Model observations also indicated that the harbor-side armor unit protection should not extend above the breakwater crown elevation any more than absolutely necessary. Tests indicated that it was preferable to leave a small gap between the concrete rib cap and the upper harbor-side armor protection than to fit a unit in this area if a large portion of the unit had to project above the crown of the structure.

PREFACE

The model investigation reported herein was initially requested by the U. S. Army Engineer Division, Pacific Ocean (POD), in a letter to the U. S. Army Engineer Waterways Experiment Station (WES) dated 15 December 1980. Funding authorizations by POD were granted in POD Intra-Army Order PODSP-CIV-81-33 and its Change Order No. 1, dated 13 February 1981 and 14 July 1981, respectively.

Model tests were conducted at WES during the period February 1981 through July 1981 under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, Dr. R. W. Whalin, Chief of the Wave Dynamics Division, and Mr. D. D. Davidson, Chief of the Wave Research Branch. Tests were conducted by Mr. D. G. Markle, Hydraulic Research Engineer, assisted by Mr. M. S. Taylor, Engineering Technician, and Mrs. B. J. Wright, Engineering Aid. This report was prepared by Mr. Markle.

Liaison was maintained during the course of the investigation by means of conferences, progress reports, and telephone conversations.

Commanders and Directors of WES during the conduct of this study and the preparation and publication of this report were COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
feet	0.3048	metres
miles (U.S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
tons (2,000 lb, mass)	907.1847	kilograms

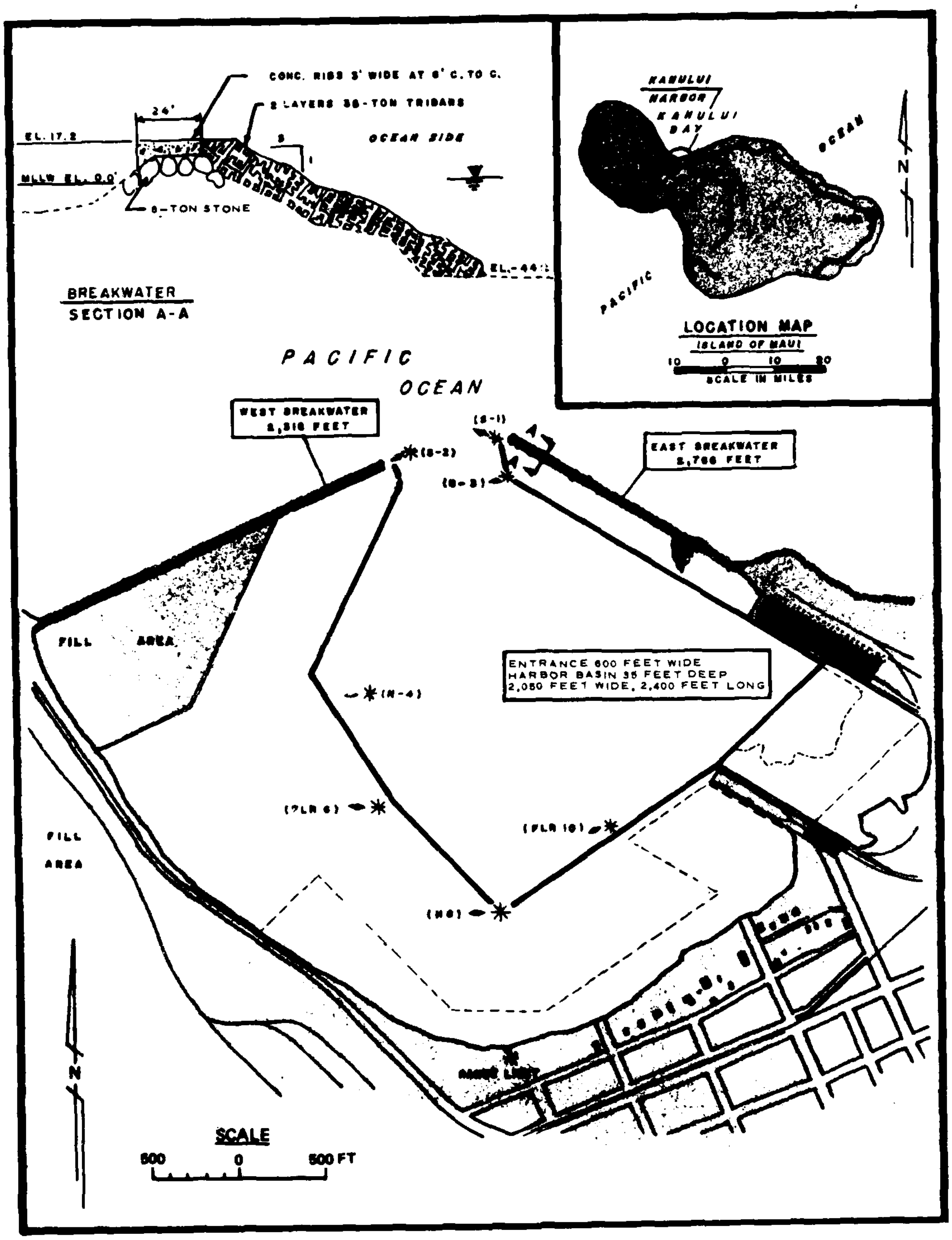

Figure 1. Kahului Harbor, Kahului, Maui, Hawaii

KAHULUI BREAKWATER STABILITY STUDY
KAHULUI, MAUI, HAWAII

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Kahului Harbor is located on the north coast of the Island of Maui (Figure 1). Kahului, Hawaii, is about 94 miles* southeast of Honolulu, Oahu, Hawaii. The harbor is protected by two rubble-mound breakwaters, the 2,766- and 2,315-ft east and west breakwaters, respectively, which were completed in 1931. Since their completion, the breakwaters have accrued significant damage during several major storms and have been repaired on numerous occasions using various sizes of dolosse, tribars, and tetrapods. Concrete caps and/or concrete ribs have also been added to the heads and portions of the trunks on both breakwaters. To date, most repair work has been concentrated on the crowns, heads, and sea-side slopes. The harbor-side slopes have degraded to very steep slopes along various lengths of both breakwaters, and the U. S. Army Engineer Division, Pacific Ocean (POD), is considering rehabilitation of these areas along with a portion of sea-side slope on the west breakwater.

Purpose of Model Study

3. The purposes of this breakwater stability study were as follows:

  a. Evaluate the stability of four proposed harbor-side rehabilitation designs, one each for the west breakwater at

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

sta* 21+25 and sta 18+50 and the east breakwater at sta 26+10 and sta 23+35.

b. Evaluate the stability of the proposed sea-side rehabilitation design for the west breakwater at sta 18+50.

c. If any of the proposed designs prove unstable for the selected test waves and still-water level conditions, test alternative designs until a stable section is found.

d. Evaluate the stability of the existing tribar protection on the sea-side slope of the west breakwater at sta 21+25.

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* For convenience, symbols and unusual abbreviations are listed and defined in the *Notation* (Appendix A).

PART II: THE MODEL

Design of Model

3. Tests were conducted at undistorted linear scales of 1:33, 1:36, and 1:40, model to prototype. Scale selections were based on the size of model armor units available relative to the size of prototype armor units existing on and/or proposed to be added to the prototype breakwaters, elimination of stability scale effects,* and capabilities of the available wave tank. Based on Froude's model law** and the linear scales of 1:33, 1:36, and 1:40, the following model-to-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristic	Dimension	Model-Prototype Scale Relations for Model Scales of 1:33	1:36	1:40
Length	L	$L_a = 1:33$	1:36	1:40
Area	L^2	$A_a = L_a^2 = 1:1,089$	1:1,296	1:1,600
Volume	L^3	$V_a = L_a^3 = 1:35,937$	1:46,656	1:64,000
Time	T	$T_a = L_a^{1/2} = 1:5.7$	1:6	1:6.3

4. The specific weight of water used in the model was assumed to be 62.4 pcf and that of seawater is 64.0 pcf. Specific weights of model breakwater construction materials were not identical to their prototype counterparts. These variables were related using the following transference equation:

$$\frac{(W_r)_m}{(W_r)_p} = \frac{(\gamma_r)_m}{(\gamma_r)_p} \left[\frac{L_m}{L_p}\right]^3 \left[\frac{(S_r)_p - 1}{(S_r)_m - 1}\right]^3 \tag{1}$$

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* R. Y. Hudson. 1975 (Jun). "Reliability of Rubble-Mound Breakwater Stability Models," Miscellaneous Paper H-75-5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

** J. C. Stevens, et al. 1942. "Hydraulic Models," Manual of Engineering Practice No. 25, American Society of Civil Engineers, New York.

where

subscripts m and p = model and prototype quantities, respectively

W_r = weight of an individual armor unit or stone, lb

γ_r = specific weight of an individual armor unit or stone, pcf

L_m/L_p = linear scale of the model

S_r = specific gravity of an individual armor unit or stone relative to the water in which the breakwater was constructed, i.e., $S_r = \gamma_r/\gamma_w$

γ_w = the specific weight of water, pcf

Test Facilities and Equipment

5. All tests were conducted in a 100-ft-long, 5-ft-wide, and 3-ft-deep flume that is located within an L-shaped wave basin, which has overall dimensions of 250 ft long, 50 and 80 ft wide at the top and bottom of the L, respectively, and 4.5 ft deep (Figure 2). The test facility is equipped with a flap-type wave generator capable of producing monochromatic waves of various periods and heights.

Model Construction and Testing Procedures

Modeling local bathymetry

6. A 1V-on-100H slope was selected as representative of the prototype bathymetry seaward of the east breakwater at sta 23+35. The bathymetry seaward of the other three test sections was represented by a 1V-on-27H slope. A 35-ft (model) length of 1V-on-27H slope was preceded by 43 ft (model) of 1V-on-100H slope in the test flume as shown in Figure 2.

Flume calibration

7. Following molding of the local bathymetry and prior to installation of the first test section, the test flume was calibrated for the wave periods and water depths selected for this study. Test waves of the required characteristics were generated by varying the frequency and amplitude of the wave generator paddle. Changes in water-surface

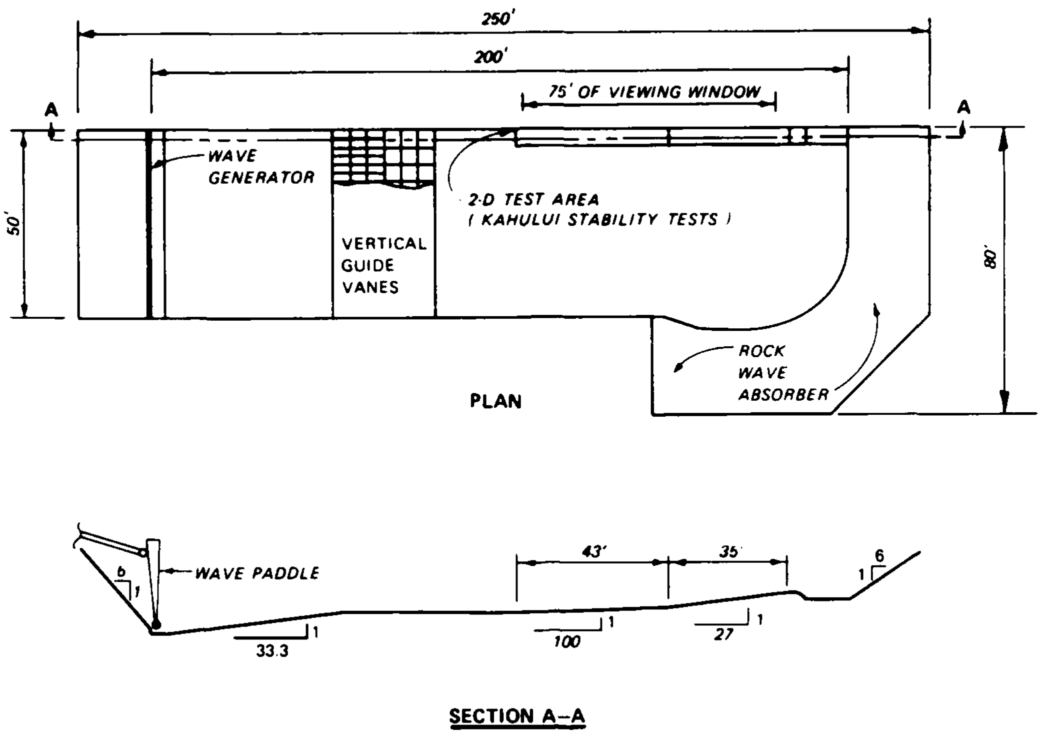


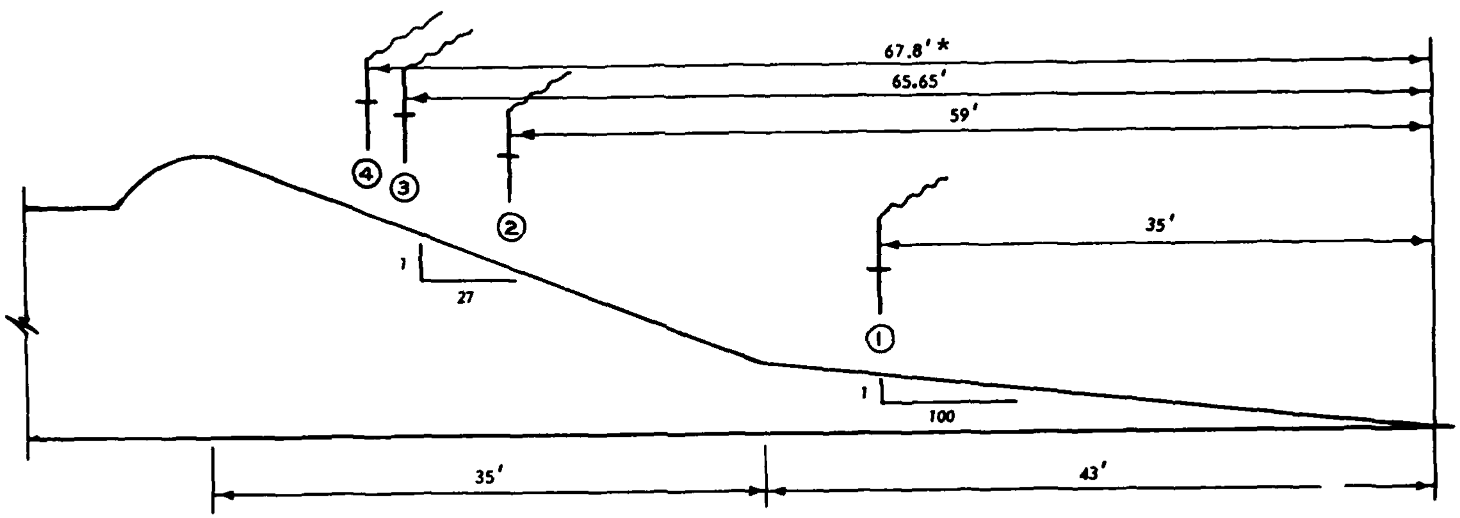
Figure 2. Test flume geometry

elevations as a function of time were measured by electrical wave-height gages and recorded on chart paper by an electrically operated oscillograph. Wave gages were located along the test slopes where the sea-side toes of the test sections would be located. Figure 3 shows the wave gage locations on the 1V-on-27H and 1V-on-100H slopes for the various test sections.

Methods of constructing test sections

8. Existing conditions of the prototype breakwater sections and characteristics of the proposed rehabilitation work were defined by means of aerial photographs and/or line drawings furnished the U. S. Army Engineer Waterways Experiment Station (WES) by POD. In cases where the proposed rehabilitation proved unstable, modifications to the new construction were made based on model observation and discussions between WES and POD personnel.

9. Model breakwater sections were constructed to reproduce, as



① EAST BREAKWATER AT STA 23+35
② EAST BREAKWATER AT STA 26+10
③ WEST BREAKWATER AT STA 21+25
④ WEST BREAKWATER AT STA 18+50
* ALL DIMENSIONS IN MODEL TERMS.

Figure 3. Wave gage locations during flume calibration

closely as possible, the existing breakwater conditions and the results of the usual methods of constructing prototype structures. Core material was dumped by bucket or shovel into the flume and was smoothed to grade and compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype breakwater. The old cover-layer stone, 16,000 lb on all test sections, was then added by shovel and smoothed to grade by hand or with trowels but was not compacted. Concrete armor units used in the cover layer, or layers, were placed either in a random manner, i.e., placed in such a way that no intentional interlocking of the units was obtained, or with uniform placement where very close spacing and some intentional interlocking of the units was achieved. (Uniform placement should not be confused with pattern placement where each unit is laid down with a predetermined orientation.) Some "special" and "extra-special" placement of the toe dolosse was used. These toe construction techniques are described and illustrated in latter portions of this report.

10. Where crown protection was provided by cast-in-place concrete caps and/or concrete ribs, it was assumed that they are or will be stable in the prototype; therefore, it was not necessary that they be dynamically similar to the prototype. The model ribs and caps, constructed of Plexiglas, were geometrically similar to their prototype counterparts and were held in place in the model, thus ensuring proper transmission, reflection, and dissipation of wave energy and the assumed stability of the structures.

Selection of test conditions

11. Based on anticipated prototype conditions and available prototype data, POD decided that the stability tests should use wave periods of 14, 16, and 18 sec. All test sections were tested with a still-water level (swl) of +4.0 ft mllw and the west breakwater sections were also tested for a low water condition of -1.0 ft mllw. This low swl was used to evaluate stability of the existing and proposed lower, sea-side slopes of the west breakwater at sta 21+25 and sta 18+50, respectively.

12. When the first test section for each of the four proposed

designs was installed in the flume, it was exposed to a range of wave heights at each of the selected wave periods and swl's. Model observations for all four initial test sections indicated that wave attack with the 14-sec wave period was significantly less severe than either the 16- or 18-sec periods. The wave forms for the 16- and 18-sec wave periods appeared to be similar with the 18-sec period being slightly more severe. Based on these observations, the 16- and 18-sec wave periods were selected for all the full length stability tests.

13. Except for the 16-sec period on the west breakwater at sta 21+25, the depth-limited breaking waves were found to be the most severe condition with respect to stability of both the sea- and harbor-side slopes for all test sections. On the west breakwater at sta 21+25, a wave height slightly less than the depth-limited breaking wave height for the 16-sec wave period was found to be more severe for the harbor-side slope. Based on these observations and those described in paragraphs 11 and 12, the following hydrographs were selected for testing the various breakwater sections: (a) Hydrograph A, Plate 1 and Table 1, for the west breakwater at sta 21+25, (b) Hydrograph B, Plate 2 and Table 2 for the west breakwater at sta 18+50, (c) Hydrograph C, Plate 3 and Table 3 for the east breakwater at sta 26+10, and (d) Hydrograph D, Plate 4 and Table 4 for the east breakwater at sta 23+35.

14. All the waves included in Hydrographs A, B, and C are best classified as depth-limited plunging breakers except for the shakedown waves which are used to simulate wave attack during construction. The test waves of Hydrograph D were depth-limited spilling breakers which appear to be a less severe form of breaking waves. These differences in wave forms for the same prototype wave periods were a result of the different bathymetry seaward of the test sections. The steeper 1V-on-27H slope was used with Hydrographs A, B, and C and produced the more severe plunging breakers while the 1V-on-100H slope was used with Hydrograph D and produced the less severe spilling breakers.

<u>Model operation</u>

15. Once test conditions for the breakwater section were experimentally determined, the breakwater cover layers were rebuilt. Before

test photographs were taken, the flume was flooded to the appropriate depth; and the structure was exposed to the shakedown and test wave conditions. Prototype test time was accumulated in 30 sec (model time) cycles, i.e., the wave generator was started, run for 30 sec, and then stopped. This procedure eliminated contamination of generated waves by reflections from the structure that could be rereflected from the wave generator. After each 30-sec cycle, sufficient time was provided for the test basin to still out before the next cycle was run. During stilling time between cycles, detailed model observations of the structure's response to the previous cycle of test waves were recorded by the model operator. These observations included any movement occurring on the structure and a general statement of the condition of the structure at that point in the test. At the conclusion of the test, the flume was drained and the after-test condition of the structure was summarized in the test notes and documented with photographs. For most of the test sections, the cover layers were then rebuilt and the test was repeated. The purpose of the repeat test was to determine if there were any uncontrolled variations in model construction technique that affected stability of the structure. In all cases the repeat tests showed very similar results.

Methods of reporting model observations and test results

16. The following list of adjectives, in order of increasing severity, was used for recording model observations and reporting test results for each test section: (1) slight, (2), minor, (3) moderate, (4) significant, (5) major, and (6) extensive. Slight and minor were used to describe acceptable results, moderate described borderline acceptability, while significant to extensive described unacceptable conditions of increasing severity. Use of these adjectives allows for some quantification of the severity and/or amount of rocking in place, onslope displacement, offslope displacement, and resulting damage accrued by the breakwater's primary cover-layer units. By using the descriptive adjectives and the before- and after-test photographs, comparisons can be made between alternative test sections.

PART III: DESCRIPTION OF EXISTING SECTIONS, TEST PLANS, AND TEST RESULTS

West Breakwater at Sta 21+25

17. The existing section, Plate 5, consists of 4,000- to 8,000-lb core stone overlaid by one layer of random-placed 16,000-lb stone. Two layers of random-placed, 38,000-lb tribars cover the sea-side slope from the -25.0 ft mllw toe to the +16.3 ft mllw crown. The 19.5-ft-wide crown is protected by cast-in-place concrete ribs. The individual ribs are 27 ft long, 4 ft high (average), and 3 ft wide. The ribs make a 46-deg angle with respect to the longitudinal axis of the breakwater and are spaced on 6-ft centers.

18. Plan 1, Plate 6 and Photos 1-3, consisted of existing conditions, except for one minor modification, and incorporated the proposed harbor-side rehabilitation design. The existing 38,000-lb tribars were represented in the model as 38,220-lb tribars. This minor difference in weight was due to the available model armor unit sizes and the selected scale necessary to model the 13,600- to 18,400-lb harbor-side armor stone proposed for the rehabilitation work. The harbor-side armor stone was randomly placed on a 1V-on-2H slope from the 16,000-lb stone apron to an elevation of +15.3 ft mllw. During Hydrograph A, Plan 1 accrued no damage to the sea-side tribars and significant damage to the harbor-side, 13,600- to 18,400-lb armor stone. During Steps 3-5, 21 armor stones were displaced from the 1V-on-2H slope. The displacement was caused by a combination of wave energy transmitted through and overtopping the structure. Minor rocking of one or two of the sea-side tribars was noted throughout the test, but no displacement occurred. By the end of Step 5, all damage had ceased and the structure sustained no further damage during the remainder of Hydrograph A. Photos 4-6 show the condition of Plan 1 after its first testing with Hydrograph A. Plan 1 was rebuilt and exposed to Hydrograph A once again. Results of the second test were very similar to the first, with 15 armor stones displaced from the 1V-on-2H, harbor-side slope and minor rocking of 2 or

3 sea-side tribars throughout the test. All damage had stabilized by the end of Step 5 and the after-test condition of Plan 1 is shown in Photos 7-9. Both testings of Plan 1 showed more damage to the harbor-side armor stone than would be acceptable for a no-damage design.

19. In an effort to stabilize the 13,600- to 18,400-lb armor stone, the harbor-side slope was flattened to 1V on 2.5H, Plan 1-1, Plate 7. During testing of Plan 1, it was noted that Steps 3 and 5 produced the most damaging conditions on the harbor-side armor stone stability. Based on this observation, it was decided that Plan 1-1 would only be exposed to the shakedown and Steps 1, 3, and 5 of Hydrograph A. After exposure to these conditions, damage to Plan 1-1 was similar to that for Plan 1.

20. Due to the limited availability of stones larger than 16,000 lb and the probability that the 16,000-lb stone would be more expensive than originally expected, POD requested that WES pursue a series of tests to determine the sizes of dolosse and tribars that were needed for stability when placed on the 1V-on-2H harbor-side slope and exposed to the wave and swl conditions of Hydrograph A.

21. Plan 1A, Plate 8 and Photos 10-12, was identical to Plan 1, except for the armor and fill material placed on the harbor side of the breakwater. Two layers of 3,740-lb dolosse were placed on a 1V-on-2H slope over the 800-lb stone used as fill between the dolosse and existing 16,000-lb stone. Random dolos placement was used on both the toe and the slope. The dolosse showed no instability during Steps 1 and 2, but Steps 3-5 caused major failure. Steps 6 and 7 caused no additional damage. As with Plans 1 and 1-1, the existing sea-side tribars showed no instability for Plan 1A. Photo 13 shows the condition of the dolosse at the end of Step 3 and Photos 14-16 show Plan 1A at the end of the test. As shown in Photo 16, a major portion of the dolosse were displaced from the 1V-on-2H slope resulting in a large area of exposed 16,000-lb stone. The dolos displacement had not subsided at the end of Step 5, and more extensive dolos damage would most likely have occurred if the duration of Step 5 were extended.

22. The dolos armor size was increased to 6,765-lb for Plan 1B,

Plate 8 and Photos 17 and 18, in an effort to find a stable dolos design. Using random placement, two layers of dolosse were placed on a 1V-on-2H slope over the 1,350-lb fill material. Plan 1B was exposed to Steps 1-5 of Hydrograph A. Less dolos displacement occurred for Plan 1B than for Plan 1A; however, the displacement was still significant and was much more than would be acceptable for a no-damage design. All the dolos damage observed occurred during Steps 3-5 and the damage had not subsided at the end of Step 5. A total of 21 dolosse had been displaced completely off the 1V-on-2H slope and there were several areas where only one layer of dolosse remained on the slope. Minor rocking of two to three tribars was noted on the sea-side slope, but no displacement was observed. Photos 19-21 show the condition of Plan 1B at the end of the test.

23. Plan 1B-1, Plate 8, was identical to Plan 1B except for the placement of the dolosse along the toe of the harbor-side 1V-on-2H slope. Plan 1B used totally random placement, whereas a "special placement" was used on the toe of Plan 1B-1. Photo 22 shows a comparison of random and special placement of toe dolos units. (In an effort to expedite determination of acceptable dolos and tribar designs for the west breakwater at sta 21+25, only Steps 1, 3, and 5 of Hydrograph A were selected for testing the stability of various designs and only one testing of each design was conducted. Once acceptable designs were found, POD selected one dolos and one tribar design and a repeat test was conducted for these plans using the full duration and all the test conditions of Hydrograph A.) One dolos was displaced during Step 3 and 18 more were displaced during Step 5 of Hydrograph A. All displacement had subsided by the conclusion of Step 5 and the after-test condition of the section is shown in Photos 23 and 24. Although major failure did not occur, the dolos displacement was significant enough for the design to be considered unacceptable.

24. For Plan 1C, Plate 9, the harbor-side dolos size was increased to 9,670 lb and no underlayer was used between the dolosse and the 16,000-lb rock. Special placement was used on the dolos toe and the remainder of the slope was constructed using two layers of randomly

placed dolosse. At the conclusion of the test, two dolosse had been displaced off the slope and out onto the 16,000-lb rock apron. All displacement had stopped, but a significant amount of rocking in place of dolosse was still occurring. Photos 25 and 26 show Plan 1C after testing.

25. Plan 1D, Plate 9, was constructed identical to Plan 1C, but the dolos size was increased to 15,325 lb. During Steps 3 and 5 of Hydrograph A, three areas of the specially placed toe were damaged and this resulted in displacement of 14 dolosse. The displacement had stopped at the end of the test, but the amount of damage was greater than that allowable for an acceptable design. Photos 27 and 28 show Plan 1D after testing.

26. Plan 1D-1, Plate 9, was identical to Plan 1D except for the "extra special" placement used on the dolos toe of Plan 1D-1. This extra special toe placement is shown in Photo 29. In this placement, extra care is taken to ensure that at least one, and most of the time two, dolosse are placed so as to hold the outermost toe dolosse in place. In using this type of placement, some areas of the toe were built using three layers of dolosse. During Step 5 of Hydrograph A, two dolosse were displaced and all damage had stabilized by the end of the test. Only very minor rocking in place of one or two additional dolosse was noted during the test. Photos 30 and 31 show the conditions of Plan 1D-1 after testing.

27. Tests were conducted for Plan 1D-2, Plate 10, to see if the toe dolosse placed in a trench using special placement would have equal or greater stability than the same size dolosse that were not trenched, but were placed using extra special toe placement. Except for the construction technique used on the harbor-side dolos toe, Plan 1D-2 was identical to Plans 1D and 1D-1. A trench approximately 8 to 10 ft wide was formed by removing the one layer of 16,000-lb stone. The 15,325-lb dolosse were placed along the toe using special placement, while the onslope dolosse were randomly placed. Four dolosse were displaced off the slope during Step 5 of Hydrograph A, but no movement occurred along the specially placed trenched toe. All damage had stabilized before the

end of the test and Photos 32 and 33 show the section after testing.

28. Plan 1E, Plate 11, was tested to determine if larger dolosse would be stable without the toe of the slope being trenched. Two layers of 20,945-lb dolosse were placed on the harbor-side slope using extra special placement along the toe and random placement on the remainder of the slope. One dolos was displaced during Step 5 of Hydrograph A. This dolos was originally positioned at the top of the slope next to the concrete rib and a large portion of the dolos projected up higher than the +16.3-ft mllw crown elevation. This was the only movement observed during testing of Plan 1E. Photos 34 and 35 show Plan 1E after testing.

29. Having found both low- and no-damage dolos designs for the harbor-side slope (based on one testing only), tests were initiated to find a tribar armor unit size that would be stable on the harbor-side slope when exposed to Steps 1, 3, and 5 of Hydrograph A. Plan 1F, Plate 12, was identical to all previous plans tested, except for the 1V-on-2H harbor-side slope design. One layer of uniform-placed, 13,065-lb tribars was used to protect the harbor-side slope. Stones with an average weight of 1,307 lb were used as fill between the tribars and the 16,000-lb stone. Major offslope displacement of the harbor-side tribars occurred during Steps 3 and 5. A total of 34 tribars had been displaced and the damage had not subsided when the test ended. The condition of Plan 1F at the end of the test is shown in Photos 36 and 37.

30. Plan 1F was rebuilt; however, the first row of tribar units was placed along the toe of the harbor-side slope with the base of all three legs resting on an approximately horizontal plane. The next row of tribars placed started up the 1V-on-2H slope. Plan 1F accrued as much or possibly more damage during the second testing than during the initial test. Approximately one-third of the tribars had been displaced when the test ended. Damage had not stabilized but the harbor-side slope had failed (Photos 38 and 39). The tribar damage was initiated by displacement of several of the tribars along the toe. Once these units were displaced, the upper slope unraveled at a very fast rate.

31. In an effort to stabilize the toe and thus achieve a stable

tribar design for the harbor-side slope, tests were initiated for Plan 1F-1 (Plate 13). A trench, approximately 8 to 10 ft wide, was formed by removing the one layer of 16,000-lb stone. The seaward side of the trench had a slope of 1V on 2H. This was accomplished by partial removal of 16,000-lb stone and backfilling the voids in this area with the same size, 1,307-lb rock that was used as a fill beneath the tribars on the upper portion of the slope. Where it was possible, the toe row of 13,065-lb tribars was placed in the trench with all three legs of the tribar unit resting on the bottom of the trench. Above the toe trench, one layer of tribars was uniformly placed on the 1V-on-2H slope. During Step 5 of Hydrograph A, one of the toe tribar units flipped over on its side and the unit just above it was displaced out onto the 16,000-lb rock apron. No other movement was observed during the test and all damage had stabilized well before the end of the test. Photos 40 and 41 show Plan 1F-1 after testing.

32. Plan 1G, Plate 12, used the identical construction as the second testing of Plan 1F, paragraph 30. On Plan 1G, the harbor-side tribar size was increased to 20,000 lb and 2,000-lb stones were used as fill between the tribars and 16,000-lb stone. The 20,000-lb tribars accrued significant damage during Step 5 of Hydrograph A. The damage started with displacement of several toe units which led to instability of the upper slope tribars and ultimate failure of the harbor-side slope. The damage did not stabilize and is shown in after-test Photos 42 and 43. Damage exceeded an acceptable amount.

33. The toe of the 20,000-lb tribars was trenched for Plan 1G-1, Plate 14, in an attempt to stabilize the toe and achieve a no-damage tribar design for the harbor-side slope. A 1V-on-2H slope was constructed in the 12-ft-wide trench and a portion of the upper slope by using a 2,000-lb rock fill. One layer of 20,000-lb tribars was placed, using uniform placement, from the toe to the crown of the harbor-side slope. The 20,000-lb tribars sustained no damage during testing and there was no evidence of any instability at the end of the test. Photos 44 and 45 show Plan 1G-1 after testing.

34. In the event that toe trenching cannot be achieved in the

prototype, tests were conducted for Plan 1H (Plate 15). Plan 1H was identical to Plan 1G, described in paragraph 32, except for the 20,000-lb tribar toe units which were replaced with 28,250-lb tribars. During Step 5 of Hydrograph A, four of the 28,250-lb tribar units were displaced and this caused some downslope slippage of the 20,000-lb tribars. The damage had not stabilized but was progressing at a very slow rate when the test was stopped. Photos 46 and 47 show Plan 1H after testing.

35. Plan 1I, Plate 15, was identical to Plan 1H except for the toe row of tribars on the harbor-side slope. The 28,250-lb tribars used in Plan 1H were replaced by 38,220-lb tribars for Plan 1I. Two of the toe tribars were displaced during Step 5 of Hydrograph A, but no other movement of tribar units was observed. All damage had stopped well before the end of the test and Photos 48 and 49 show Plan 1I after testing.

36. POD stated that the toe trenching could be accomplished in the prototype and selected Plans 1F-1 and 1D-2, Plates 13 and 10, respectively, for repeat testing using all seven steps of Hydrograph A. Plan 1F-1 was reconstructed, Photos 50 and 51, and exposed to the wave and swl conditions of Hydrograph A. No displacement of the 13,065-lb tribars occurred. Photos 52 and 53 show Plan 1F-1 after testing.

37. Plan 1D-2 was reconstructed in the test flume, Photos 54 and 55, and exposed to Hydrograph A. Exposure to the shakedown and Steps 1 and 2 caused no dolos movement on the harbor side of the breakwater. During the early portion of Step 3, one dolos was displaced from the upper slope onto the 16,000-lb rock apron and one dolos from the upper slope was displaced on, but not off of, the 1V-on-2H slope. No other movement occurred until the first part of Step 5. At this time, several dolosse near the center of the section and on the upper slope turned over but were not displaced from the dolos area. Approximately halfway through Step 5, one additional dolos turned over in place and the dolos that was displaced onslope during Step 3 was moved off the slope and out onto the rock apron. All movement stopped well before the end of Step 5, and no additional movement was noted during exposure to the remainder of

Hydrograph A. Photos 56 and 57 show the condition of Plan 1D-2 after this test.

38. During the first tests of the west breakwater at sta 21+25, the existing sea-side tribars proved to be stable for the test conditions of Hydrograph A. Therefore, the sea-side slope was not rebuilt between subsequent tests for the last 12 testings of various harbor-side designs. Minor rocking of one or two tribars was noted during a few of the earlier tests, but no displacement of tribars or deterioration of the sea-side slope was noted. Thus, the after-testing, sea-side view of Plan 1D-2, Photo 58, shows the 38,220-lb tribars after a cumulative exposure to approximately 4 hr of Step 1, 0.5 hr of Step 2, 12 hr of Step 3, 1 hr of Step 4, 12 hr of Step 5, 0.5 hr of Step 6, and 0.5 hr of Step 7 of Hydrograph A.

West Breakwater at Sta 18+50

39. The existing section, Plate 16, consists of 4,000- to 8,000-lb core stone overlaid with one layer of random-placed, 16,000-lb stone. The 16,000-lb stone extends from the -25.0 ft mllw sea-side toe to a crown elevation of +12.1 ft mllw and continues down the harbor side to the toe elevation of -17.0 ft mllw. Seaward of sta 18+50, the sea floor rises to a shallower depth of approximately -20.0 ft mllw. Therefore, a depth-limited breaking wave for the water depth existing at the sea-side toe of sta 18+50 would break seaward of the breakwater and the maximum breaking wave that could reach the breakwater at sta 18+50 is controlled by the -20.0 ft mllw elevation seaward of the breakwater. Based on this, the decision was made to use a -20.0 ft mllw sea-side toe elevation in the model with a 1V-on-27H slope extending seaward from the toe of the structure.

40. A concrete wall extends up to an elevation of +13.0 ft mllw on the harbor side of the crown. Portions of the wall have been lost during severe storms and the remaining wall is not structurally sound. The wall does reduce overtopping wave energy, but due to its structural weakness and the probability of destruction during future storms, it was

assumed nonexistent for purposes of the model study.

41. Plan 2, Plate 17 and Photos 59-61, was constructed to a crown elevation of +12.1 ft mllw. One layer of random-placed, 16,000-lb stone covered the 4,000- to 8,000-lb core material. The proposed rehabilitation construction consisted of one layer of uniform-placed tribars on 1V-on-2H slopes on the sea side and harbor side of the breakwater. The toes of the 21,758-lb, sea-side tribars and the 10,064-lb, harbor-side tribars were trenched into the existing 16,000-lb stone. Stone averaging 2,176 lb was used as fill between the sea-side tribars and the 16,000-lb stone. During Step 2 of Hydrograph B, several of the 16,000-lb stone were displaced from the sea-side toe up onto the lower, sea-side tribars; but this did not result in any instability of the tribar toe area. Steps 3 and 4 caused a moderate amount of displacement and reorientation of the tribars and armor stone along the breakwater crown. Six of the sea-side tribar units, four along the crown and two on the lower slope, were turned up on their sides and one additional sea-side tribar unit was displaced over the crown and down to the toe of the harbor-side tribars. Displacement of 16,000-lb stone on the breakwater crown resulted in spot lowerings of 3 to 4 ft. None of the harbor-side tribars were displaced from their original location, but a noticeable bulge occurred in one area of the upper slope. This occurred due to a combination of overtopping and transmitted wave energy and the impact of the 16,000-lb crown stone against the tribars along the top of the harbor-side slope. A few sea-side tribar units rocked in place throughout testing at the +4.0 ft mllw swl, but all damage had subsided well before the end of Step 4 and no additional displacement occurred during the remainder of Hydrograph B. Photos 62-64 show the condition of Plan 2 at the conclusion of the first test. Plan 2 was rebuilt and once again exposed to Hydrograph B. A moderate amount of reorientation and displacement occurred on the upper sea- and harbor-side slopes. None of the sea-side tribar units were displaced during this test, but three harbor-side tribar units were displaced down to the 16,000-lb stone on the harbor-side toe. Displacement of the 16,000-lb crown stone once again resulted in spot lowerings of 3 to 4 ft along the breakwater crown.

All damage had stabilized during Step 4 of Hydrograph B and the condition of the structure at the end of the test is shown in Photos 65-67.

42. A concrete rib cap was added to the crown of the breakwater to provide a buttress for the sea-side tribars, to help stabilize the crown stone, and to reduce the amount of overtopping wave energy reaching the harbor-side tribar units. To expedite the tests, the rib cap used was the same one as had been used for all of the tests previously reported for the west breakwater at sta 21+25. Due to the difference in model scales, 1:36 for sta 21+25 and 1:33 for sta 18+50, the rib cap represented a slightly smaller prototype size for the tests at sta 18+50. The individual ribs represented were 24.8 ft long, 3.7 ft high, and 2.8 ft wide. Other than the concrete rib cap and the extension of the sea-side tribars up to the +15.8 ft mllw crown elevation, Plan 2A (Plate 18 and Photos 68-70) was identical to Plan 2. Plan 2A was exposed to the wave and swl conditions of Hydrograph B. Compared with Plan 2, there was a noticeable decrease in the amount of overtopping wave energy observed in Plan 2A and only minor damage was accrued by the breakwater. Two or three 16,000-lb stones were displaced up onto the toe of the sea-side tribars and two tribars were displaced from one area of the harbor-side slope. All damage had stopped well before the end of Step 4, and Photos 71-73 show the condition of the breakwater at the end of the test. Plan 2A was rebuilt and once again exposed to Hydrograph B. One armor stone was displaced from the sea-side toe area up onto the sea-side tribars during Step 2. This was the only movement observed during the test and Photos 74-76 show that Plan 2A was in very good condition at the end of the test.

East Breakwater at Sta 26+10

43. The existing section, Plate 19, consists of 4,000- to 8,000-lb core stone overlaid by one layer of random-placed 16,000-lb stone. The sea-side slope is covered by two layers of random-placed, 70,000-lb tribars and the crown is covered by concrete ribs that overlay a concrete cap. The individual ribs are 20.0 ft long, 3.0 ft wide, and

vary in height from 4.0 ft on the sea side to 10.0 ft on the harbor side. The ribs are spaced on 6.0-ft centers, are oriented at a 90-deg angle to the longitudinal axis of the breakwater, and extend up to an elevation of +16.6 ft mllw. The elevations of the sea- and harbor-side toes are -40.0 and -28.3 ft mllw, respectively, Just seaward of the breakwater toe at sta 26+10 the sea floor rises to an elevation of -38.0 ft mllw.

44. Plan 3, Plate 20 and Photos 77-79, was constructed in the model and reproduced the existing conditions except for the following modifications: (a) the 70,000-lb sea-side tribars were represented as 70,800 lb due to size of available model units and the 1:40-scale selected to reproduce the size of tribar units tested on the harbor-side slope, and (b) the -38.0 ft mllw elevation was selected as the controlling depth for the maximum breaking wave height that could reach the breakwater; therefore, the model breakwater toe was located at this elevation. The harbor-side rehabilitation work consisted of one layer of 17,920-lb tribars, uniformly placed on a 1V-on-2H slope. The toe of the tribars was trenched into the 16,000-lb stone and the tribar protection extended up to +12.6 ft mllw. The area between the tribars and 16,000-lb stone was filled with 1,790-lb stone. Plan 3 accrued very slight damage during its first exposure to the wave conditions of Hydrograph C. The existing sea-side tribars showed some slight reorientation and two units were displaced onslope. The only other movement observed was some displacement of 16,000-lb stone on the harbor-side toe and some slight rocking in place of two or three tribar units on the upper harbor-side slope. All damage had stopped well before the end of Hydrograph C and Photos 80-82 show that Plan 3 was in excellent condition at the end of the first test. The harbor-side rehabilitation area was rebuilt and the test conditions of Hydrograph C were repeated. The test results were identical to the first test, except for the displacement of one harbor-side tribar unit that occurred during Step 1. All damage stabilized well before the end of the test. Photos 83-85 show Plan 3 after testing.

East Breakwater at Sta 23+35

45. The existing breakwater section, Plate 21, consists of 4,000- to 8,000-lb core stone overlaid by one layer of random-placed 16,000-lb stone. The sea-side slope is covered from the -45.0 ft mllw toe elevation to the +14.8 ft mllw crown with two layers of random-placed 60,000-lb dolosse. Portions of an old access road still exist on the breakwater crown; the road is contained by concrete walls on the sea- and harbor-sides of the crown. The gravel fill and asphalt cap between the walls has accrued significant damage, and stability of the walls is very questionable. Therefore (for purposes of the model study), the walls were considered nonexistent and the crown elevation of the first test sections was set at +11.5 ft mllw which is the existing elevation of the 16,000-lb stone at sta 23+35.

46. Plan 4, Plate 22 and Photos 86-88, was constructed in the model to reproduce conditions described in the previous paragraph. Due to the 1:40 scale necessary to reproduce the proposed harbor-side tribar size, the nearest available size of model dolos units represented 58,500 lb instead of the desired 60,000 lb. The harbor-side rehabilitation construction was composed of one layer of 17,920-lb tribars uniformly placed on a 1V-on-2H slope. The tribar toe was trenched into the 16,000-lb stone and the units extended up to the +11.5 ft mllw crown elevation. Stone weighing an average of 1,790 lb was used as fill between the tribars and 16,000-lb stone. During the shakedown and both steps of Hydrograph D, significant damage was accrued by the breakwater crown area. A large amount of reoriention, rocking in place, and displacement of the 58,500-lb dolosse occurred along the upper sea-side slope. Four dolosse were displaced over the crown and down onto the harbor-side slope. One additional dolos was locked into the 16,000-lb stone on the crown. The 16,000-lb crown stones were displaced both toward the back of the crown and completely over the crown and down the harbor side of the breakwater, resulting in up to 10-ft reductions in crown elevation at isolated locations. One harbor-side tribar was displaced from the upper slope down to the harbor-side stone apron. No

other tribar displacement was observed; however, a significant number of crown stones were pushed back and lodged against the top harbor-side tribars. The impact between the armor stone and tribars would most likely result in more significant prototype damage due to possible breakage of armor units. The rate of displacement and damage to the breakwater crown had slowed but had not subsided at the end of the hydrograph. Photos 89-91 show that damage sustained by the breakwater far exceeded an acceptable amount for a no-damage design.

47. A concrete rib cap was added to the section to provide buttressing for the dolosse and to help stabilize the armor-stone crown. The rib cap used in Plan 3A was modified to represent a constant 4-ft height. Plan 4 was rebuilt and with the rib cap installed was referred to as Plan 4A, Plate 22 and Photos 92-94. During exposure to Hydrograph D there was some minor in-place rocking and reorientation of the dolos and tribar units along the edges of the breakwater crown. In a repeat test, the harbor-side armor layer was rebuilt and the section was once again subjected to the wave conditions of Hydrograph D. Results of the repeat test were very similiar to results of the original test. During both tests, a significant reduction in overtopping wave energy was observed for Plan 4A relative to Plan 4. Photos 95-97 and 98-100 show the conditions of Plan 4A after the first and second tests, respectively.

PART IV: CONCLUSIONS

48. Based on the tests and results reported herein, it is concluded that:

a. For the west breakwater at sta 21+25 exposed to the wave and swl conditions of Hydrograph A, Plate 1 and Table 1:

   (1) The existing 38,220-lb (19 ton) tribars are an adequate design for the sea-side slope.

   (2) The 1V-on-2H harbor-side slopes of Plans 1, 1A, 1B, 1B-1, 1C, 1D, 1F, 1G, and 1H and the 1V-on-2.5H harbor-side slope of Plan 1-1 are not adequate designs.

   (3) Plans 1D-1, 1D-2, 1F-1, and 1I are adequate 1V-on-2H harbor-side slope designs if some potential minor damage and displacement of armor units are acceptable.

   (4) Plans 1G-1 and 1E appear to be unconditionally acceptable designs of IV-on-2H harbor-side slope rehabilitation. The harbor-side rehabilitation portions of these plans consisted of 10-ton tribars and 10.5-ton dolosse, respectively. The tribars were keyed into the existing 8-ton stone layer with a trench. For the dolosse, extra special placement of the toe units was necessary.

b. The proposed rehabilitation designs for the sea- and harbor-side slopes of Plan 2 for the west breakwater at sta 18+50 are not adequate for the wave and swl conditions of Hydrograph B, Plate 2 and Table 2. With the addition of a concrete rib cap (Plan 2A), the same armor and slope designs tested for Plan 2 proved to be adequate. The sea-side slope rehabilitation consisted of 11-ton tribars, and 5-ton tribars were used on the harbor-side slope. Both slopes were IV on 2H and were keyed into the existing 8-ton stone layer with a trench.

c. The existing sea-side slope (35-ton tribars) and the proposed harbor-side rehabilitation design for Plan 3 for the east breakwater at sta 26+10 proved to be adequate for the test conditions of Hydrograph C, Plate 3 and Table 3. The harbor-side rehabilitation consisted of 9-ton tribars on a IV-on-2H slope layered into the existing 8-ton stone layer with a trench.

d. The crown areas of the proposed harbor-side design and existing sea-side design of Plan 4 for the east breakwater at sta 23+35 were not adequate for the wave and swl

conditions of Hydrograph D, Plate 4 and Table 4. Plan 4A, identical to Plan 4 except for the addition of a concrete rib cap, proved acceptable when exposed to the identical test conditions. The harbor-side rehabilitation consisted of 9-ton tribars on a IV-on-2H slope keyed into the existing 8-ton stone layer with a trench.

PART V: DISCUSSION

49. It is imperative that the trenching and/or special placements described in this report for the toe units be strictly followed during construction. The stability of the various acceptable designs is highly dependent upon achieving the toe placement recommended. Failure to follow the guidance provided regarding toe design will probably result in significant damage to the structure.

50. Model observations indicated that during rehabilitation construction of the harbor-side slopes, care should be taken to maintain as low a profile as possible along the crown of the slope, i.e., regardless of the type (dolos or tribar) of armor unit being placed, the crown units should not project above the concrete ribs more than necessary. Model tests indicated that (for the harbor-side slopes), it was better to leave a small gap between the concrete ribs and the top armor units than to fit a unit in this space if a large portion of the unit had to project above the crown elevation of structure.

Table 1

Hydrograph A

Step	swl ft mllw	Test Wave Period sec	Test Wave Height ft	Prototype Duration hr	Wave Type
	-1.0	16.0	10.0	0.25	Shakedown
1	-1.0	16.0	19.5	0.25	Worst breaking (Sea-side armor)
2	-1.0	18.0	21.0	0.25	Worst breaking (Sea-side armor)
3	+4.0	16.0	24.5	1.0	Worst breaking (Harbor-side armor)
4	+4.0	16.0	25.5	0.5	Worst breaking (Sea-side armor)
5	+4.0	18.0	25.6	1.0	Worst breaking (Sea- and harbor-side armor)
6	-1.0	18.0	21.0	0.25	Worst breaking (Sea-side armor)
7	-1.0	16.0	19.5	0.25	Worst breaking (Sea-side armor)

Table 2

Hydrograph B

Step	swl ft mllw	Test Wave Period sec	Test Wave Height ft	Prototype Duration hr	Wave Type
	-1.0	16.0	9.0	0.25	Shakedown
1	-1.0	16.0	16.0	0.25	Worst breaking
2	-1.0	18.0	18.0	0.25	Worst breaking
3	+4.0	16.0	20.5	1.00	Worst breaking
4	+4.0	18.0	21.5	1.00	Worst breaking
5	-1.0	18.0	18.0	0.25	Worst breaking
6	-1.0	16.0	16.0	0.25	Worst breaking

Table 3

Hydrograph C

Step	swl ft mllw	Test Wave Period sec	Test Wave Height ft	Prototype Duration hr	Wave Type
	+4.0	16.0	15.0	0.25	Shakedown
1	+4.0	16.0	30.5	1.00	Worst breaking
2	+4.0	18.0	34.0	1.00	Worst breaking

Table 4

Hydrograph D

Step	swl ft mllw	Test Wave Period sec	Test Wave Height ft	Prototype Duration hr	Wave Type
	+4.0	16.0	15.0	0.25	Shakedown
1	+4.0	16.0	29.0	1.00	Worst breaking
2	+4.0	18.0	29.8	1.00	Worst breaking

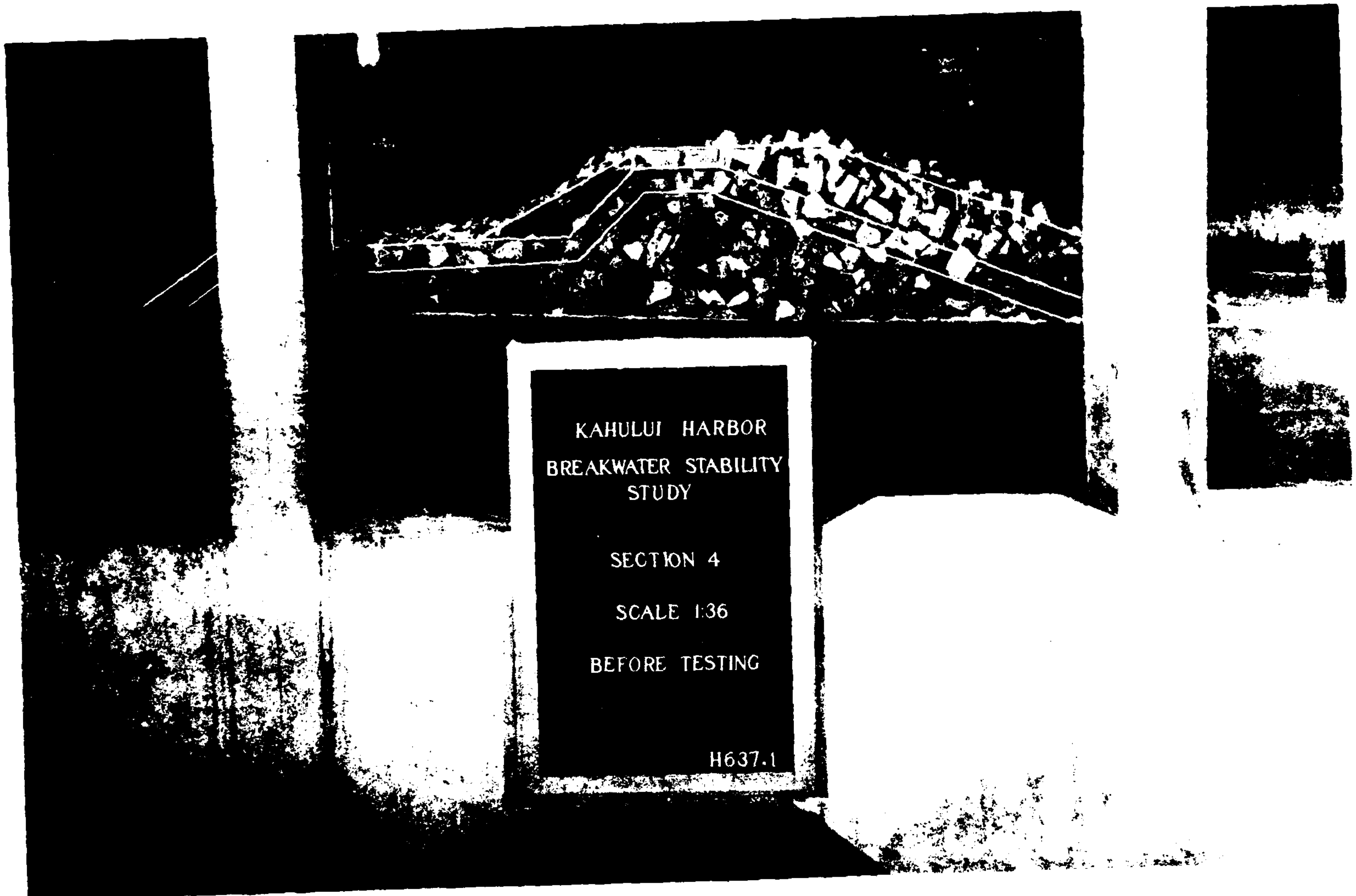


Photo 1. Side view of Plan 1 before testing

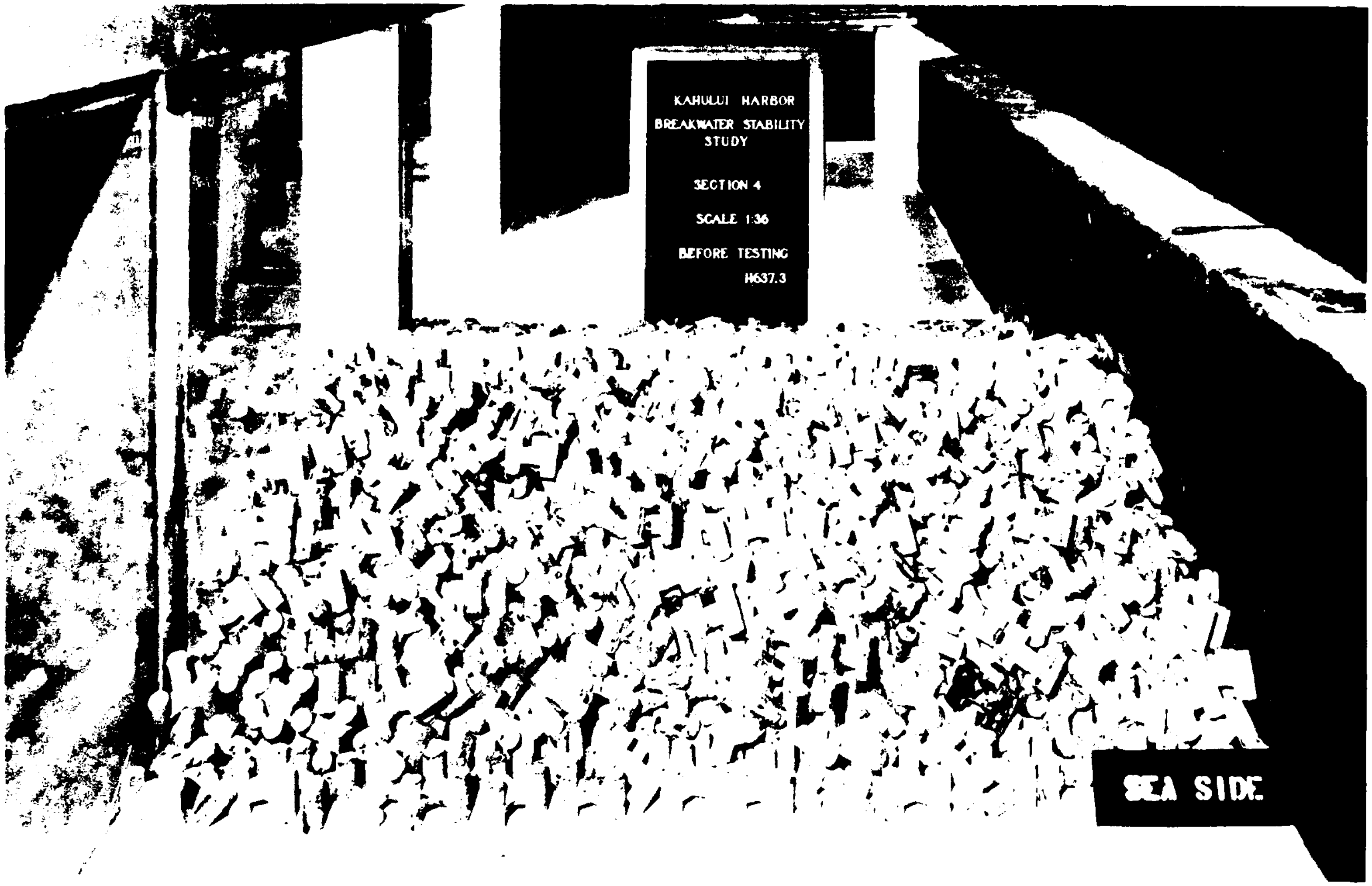


Photo 2. Sea-side view of Plan 1 before testing

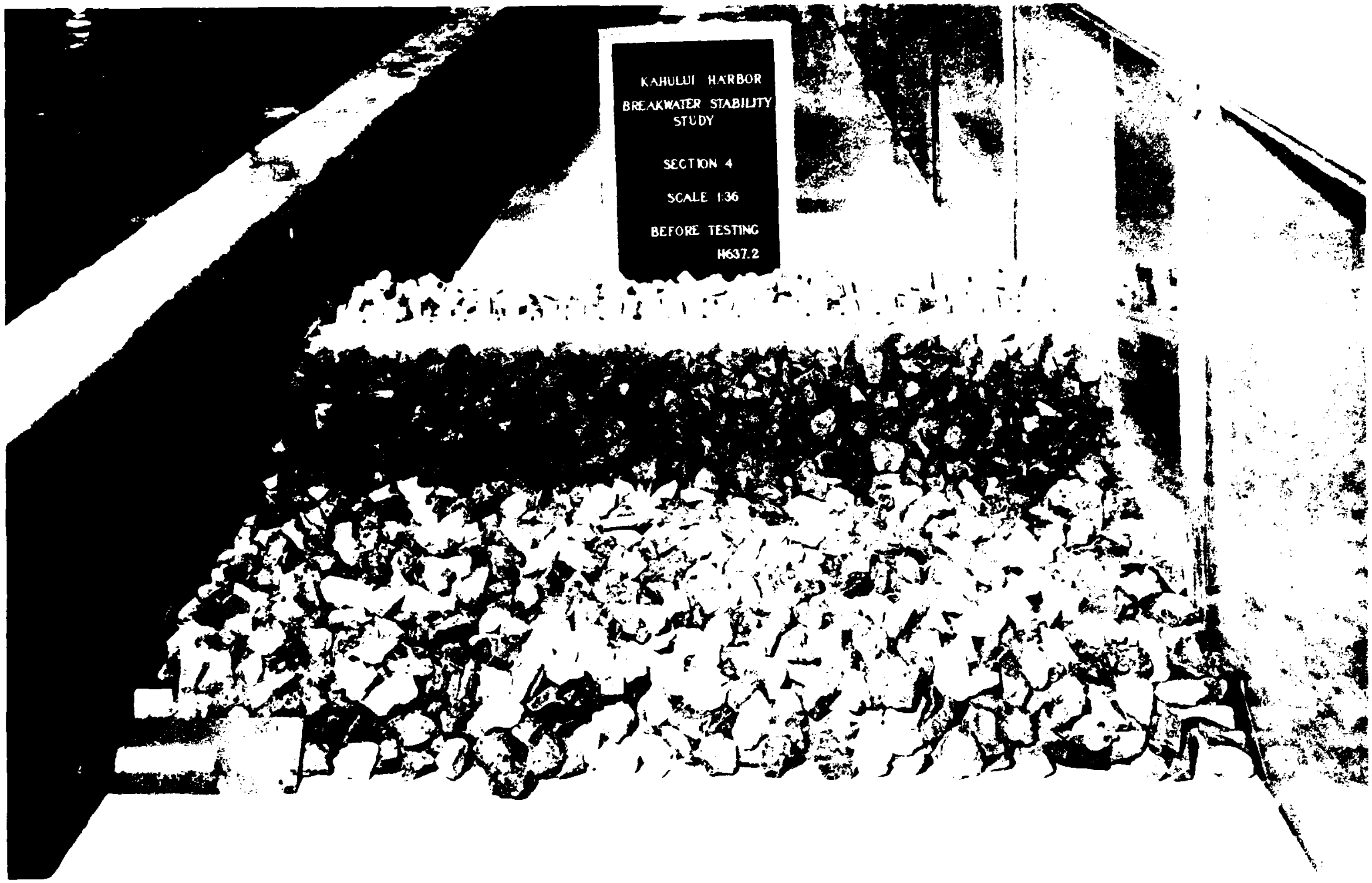


Photo 3. Harbor-side view of Plan 1 before testing

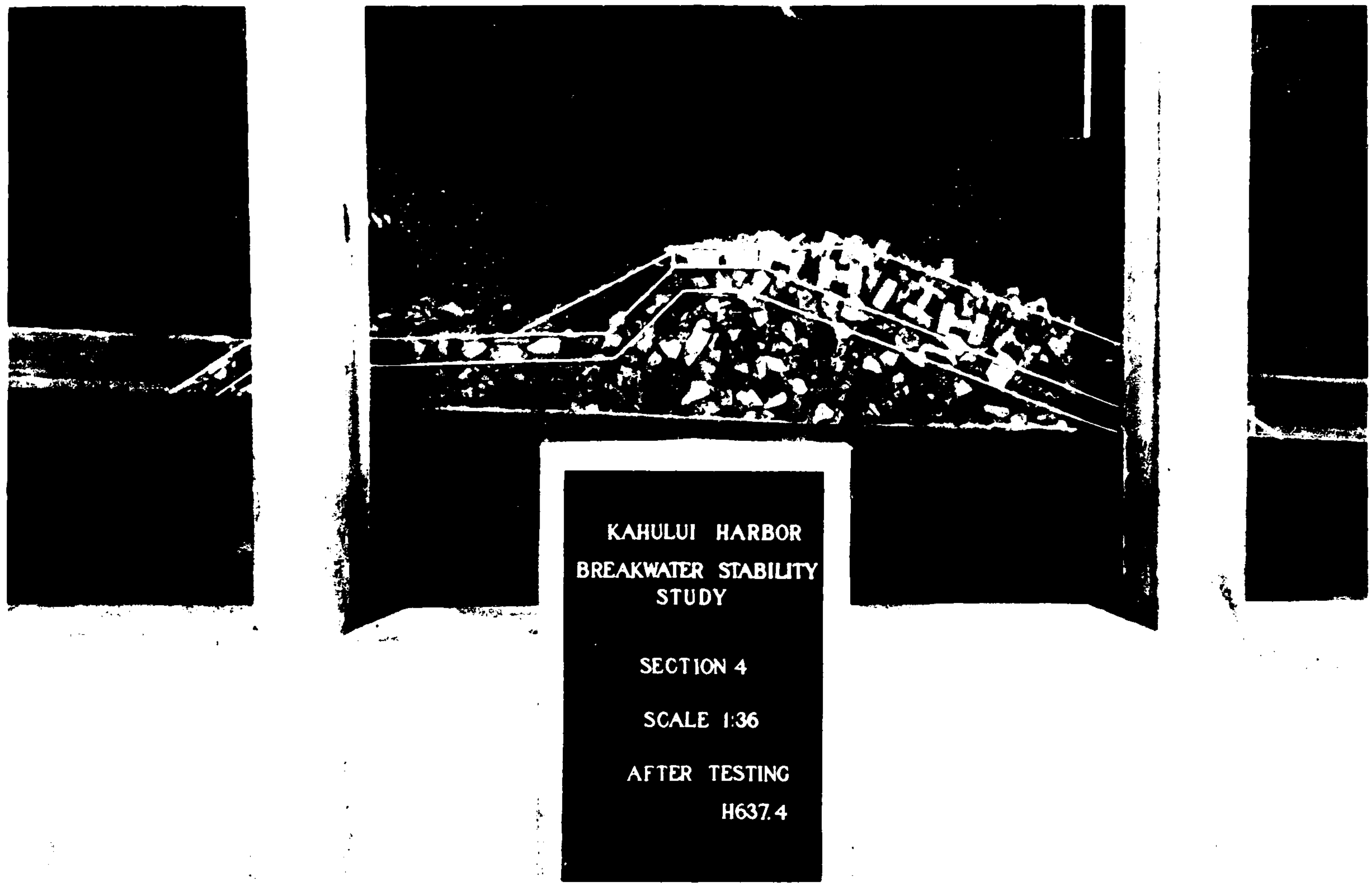


Photo 4. Side view of Plan 1 after testing, 1st test

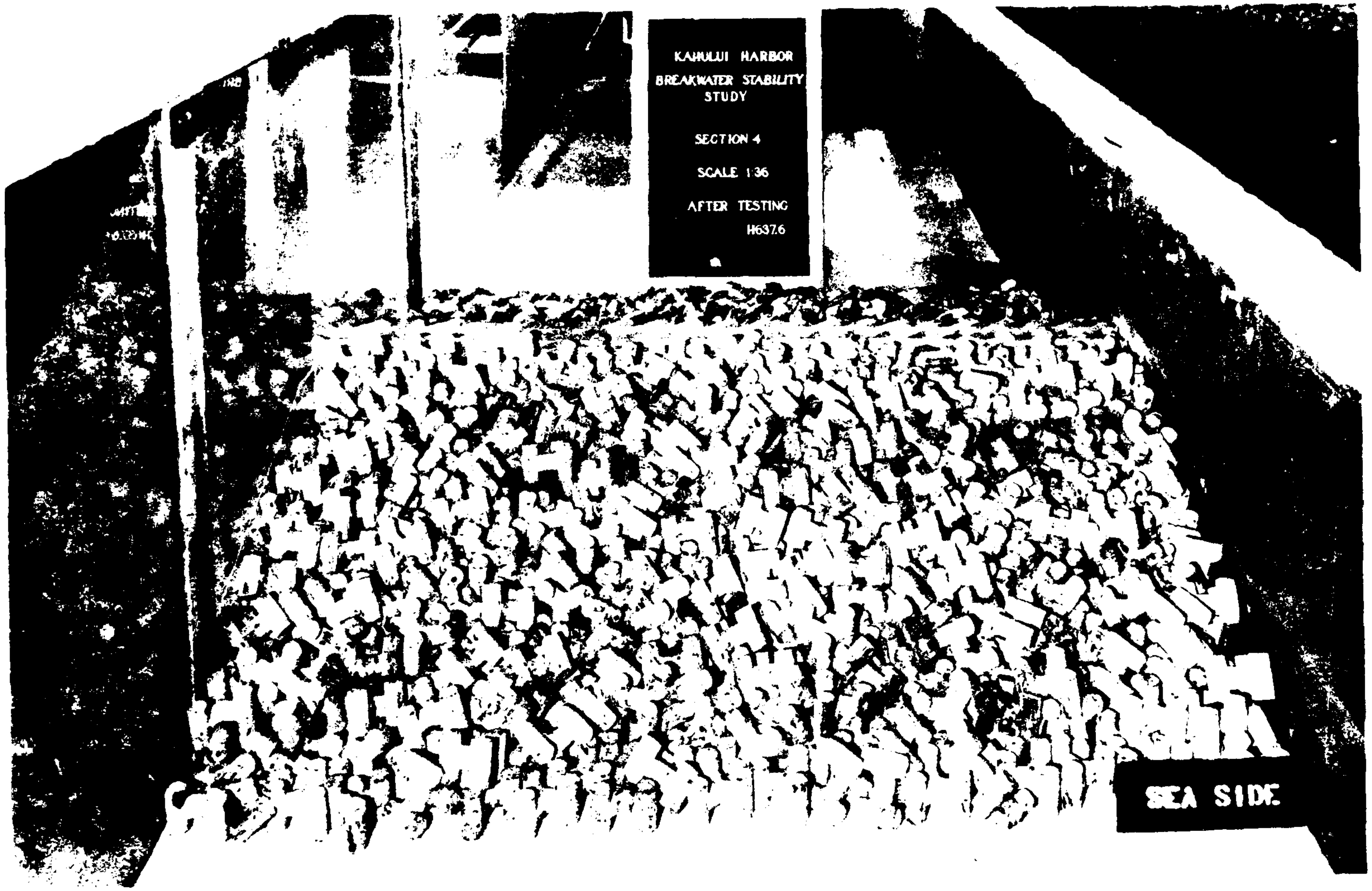


Photo 5. Sea-side view of Plan 1 after testing, 1st test

Photo 6. Harbor-side view of Plan 1 after testing, 1st test

Photo 7. Side view of Plan 1 after testing, 2nd test

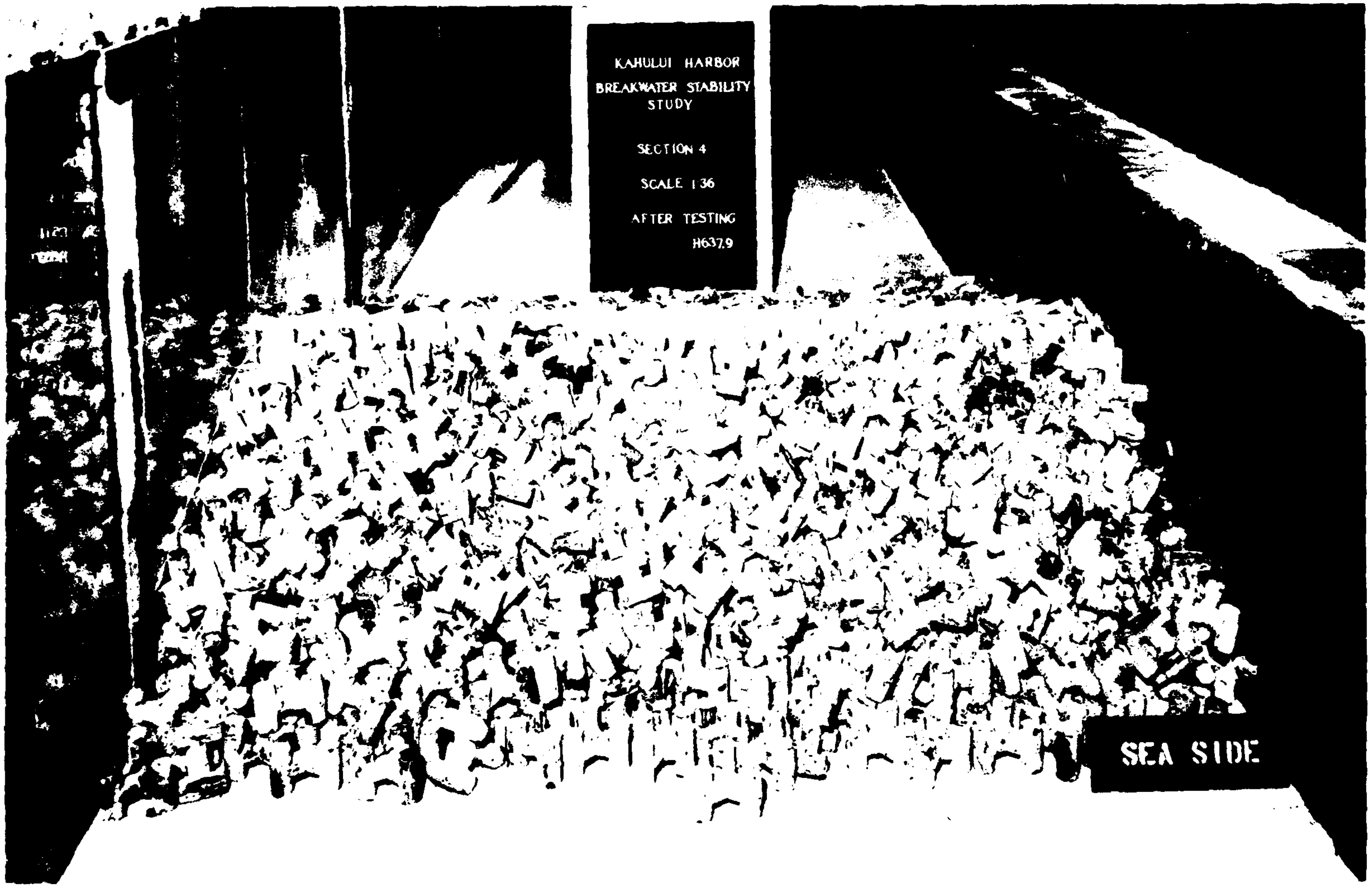


Photo 8. Sea-side view of Plan 1 after testing, 2nd test

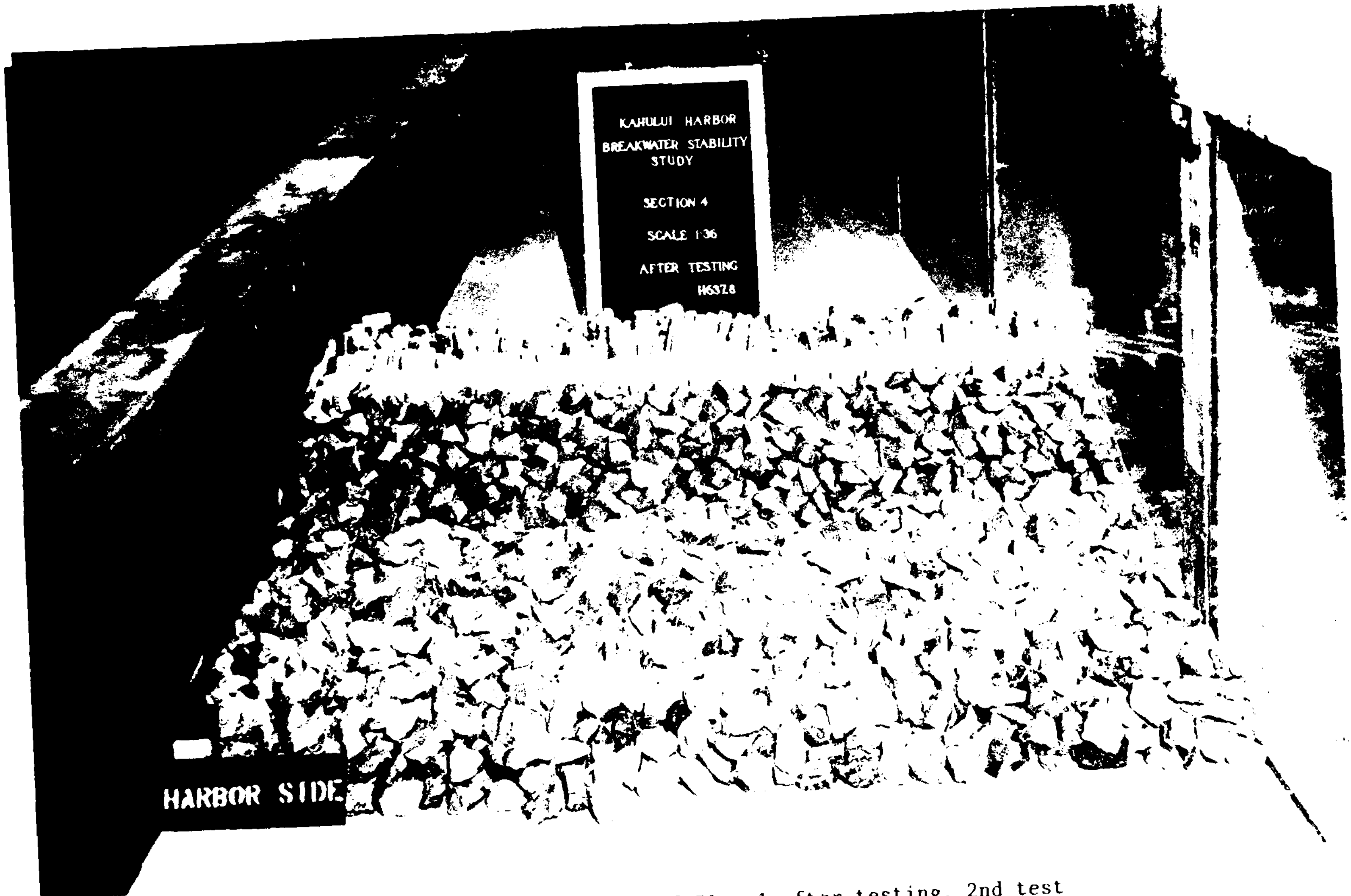


Photo 9. Harbor-side view of Plan 1 after testing, 2nd test

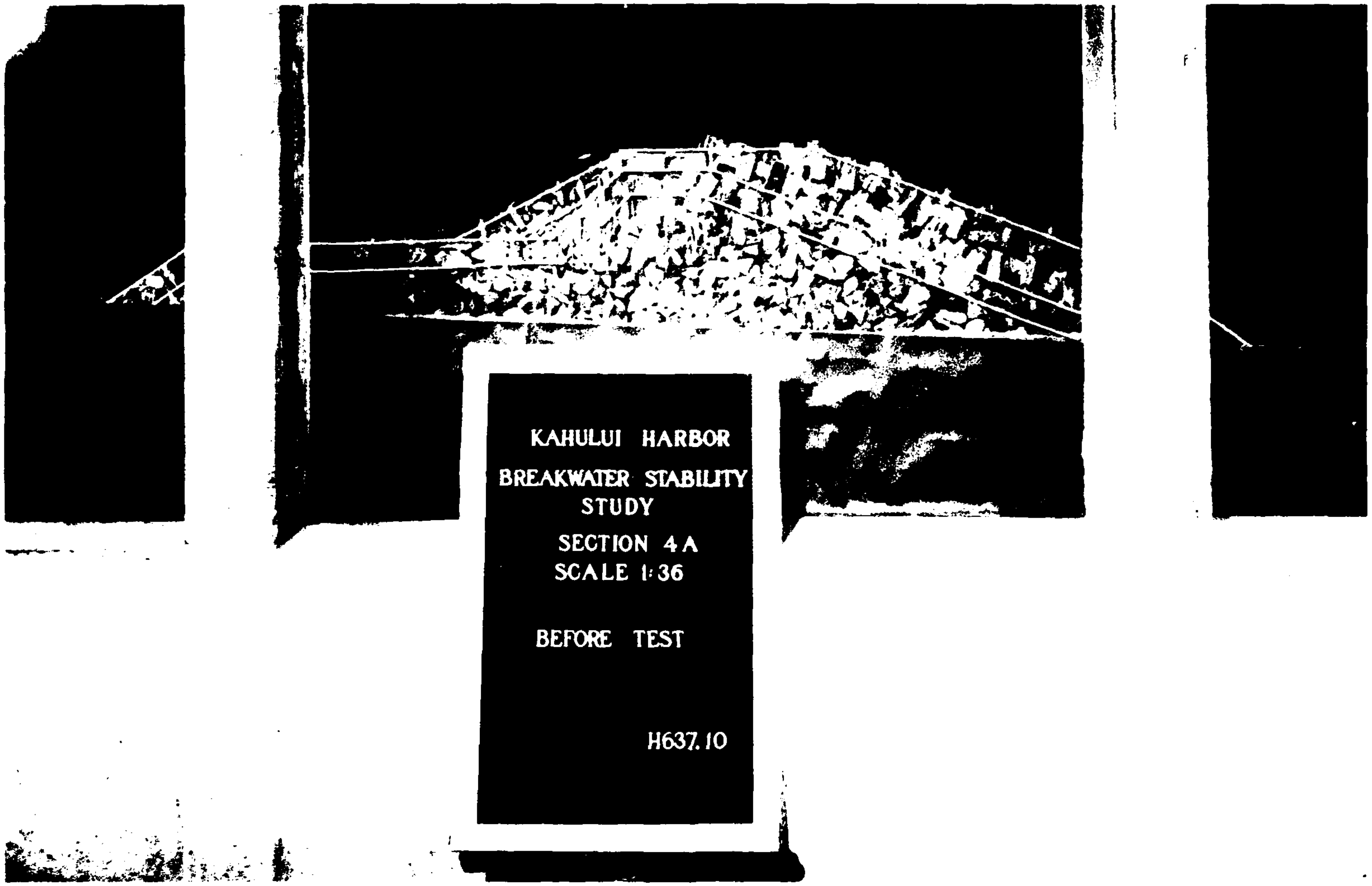


Photo 10. Side view of Plan 1A before testing

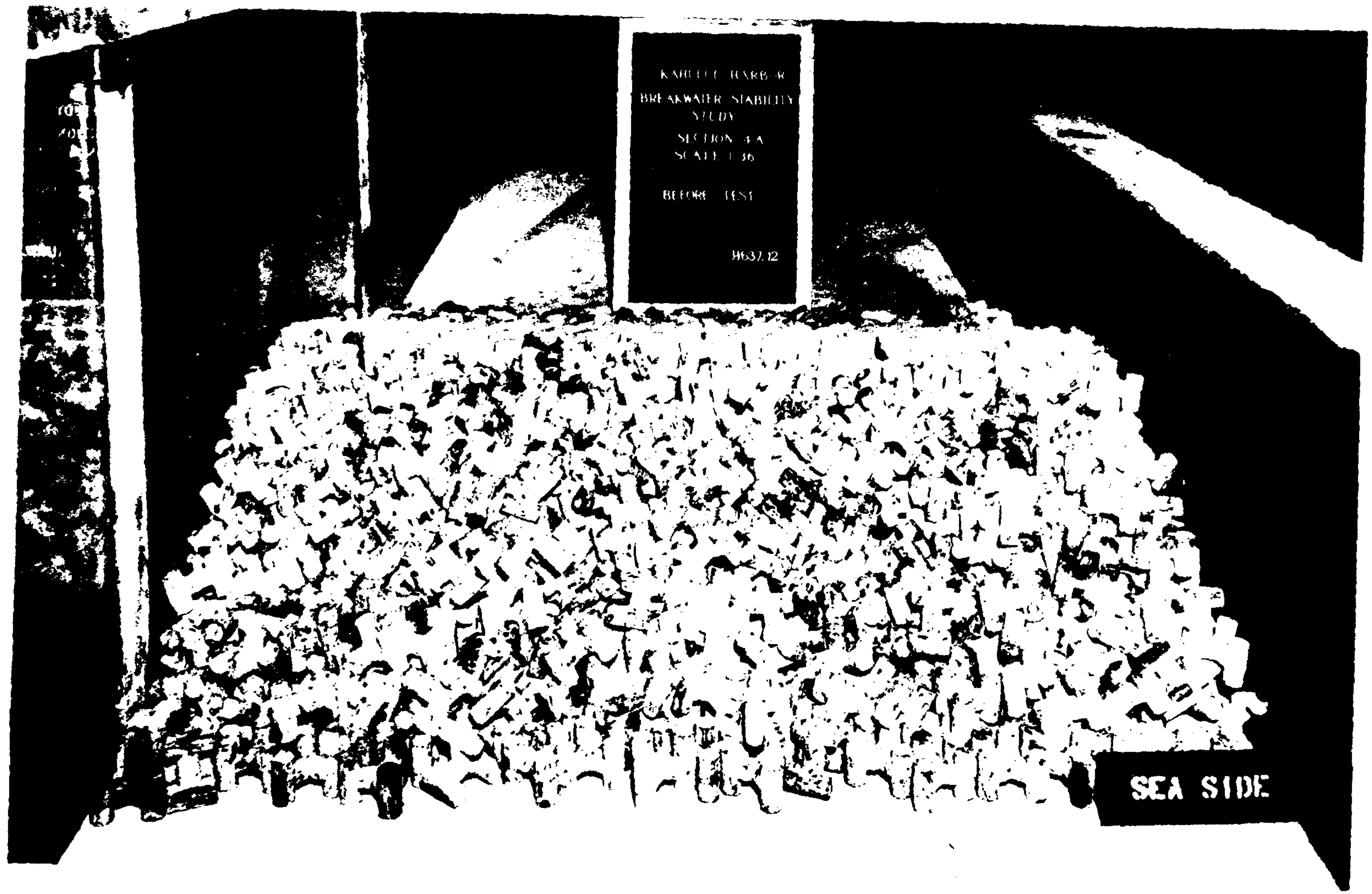


Photo 11. Sea-side view of Plan 1A before testing

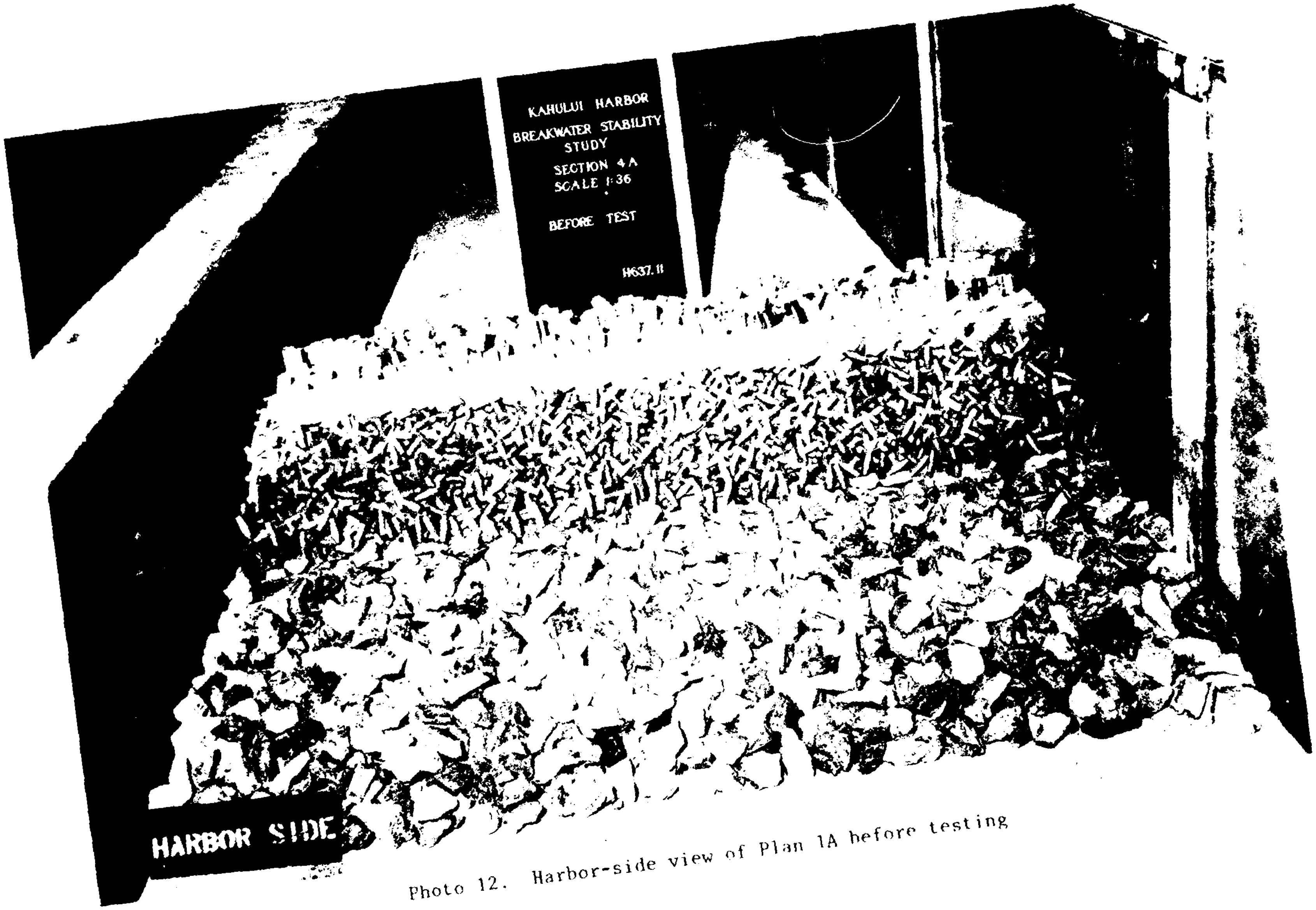


Photo 12. Harbor-side view of Plan 1A before testing

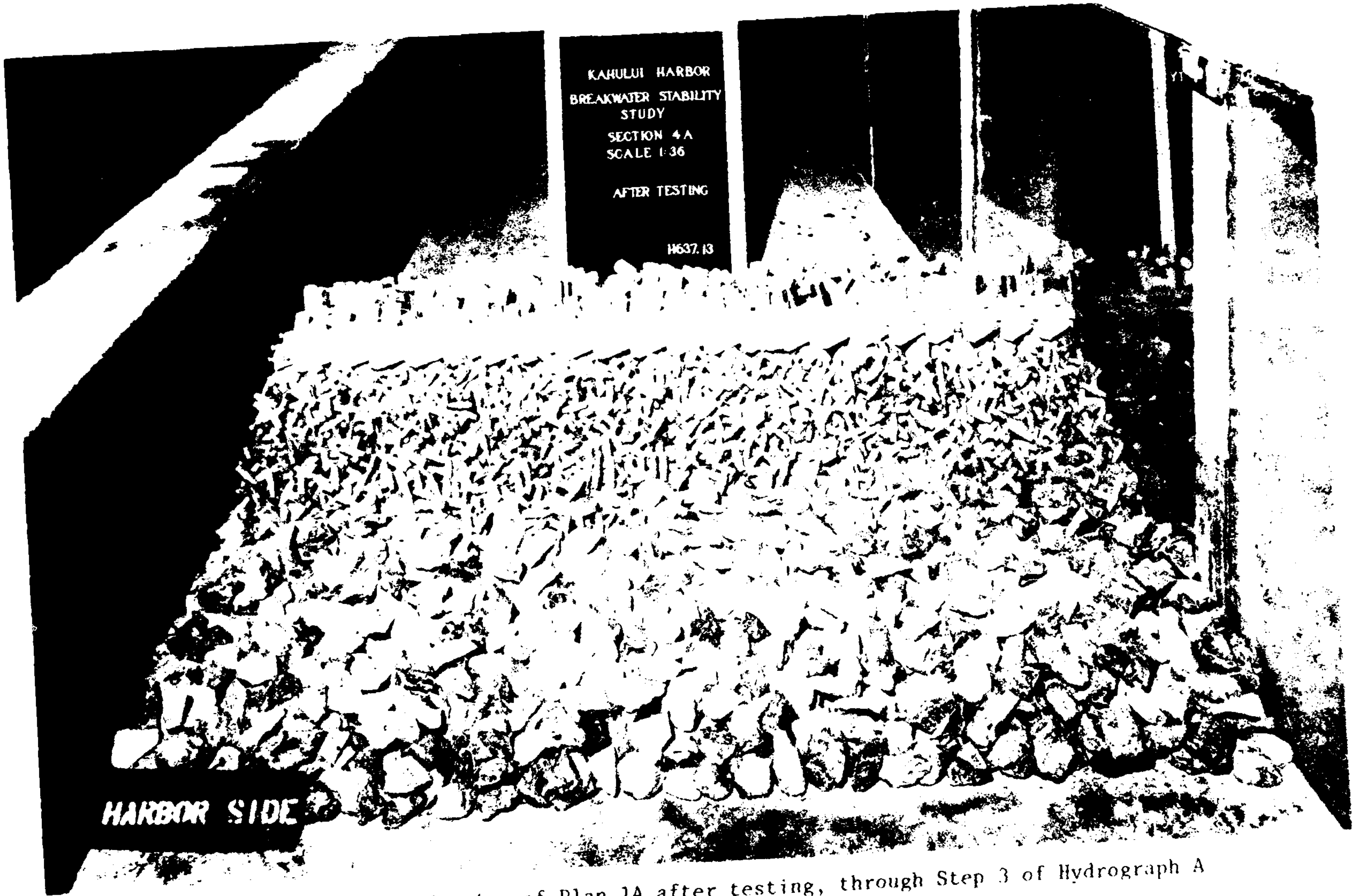


Photo 13. Harbor-side view of Plan 1A after testing, through Step 3 of Hydrograph A

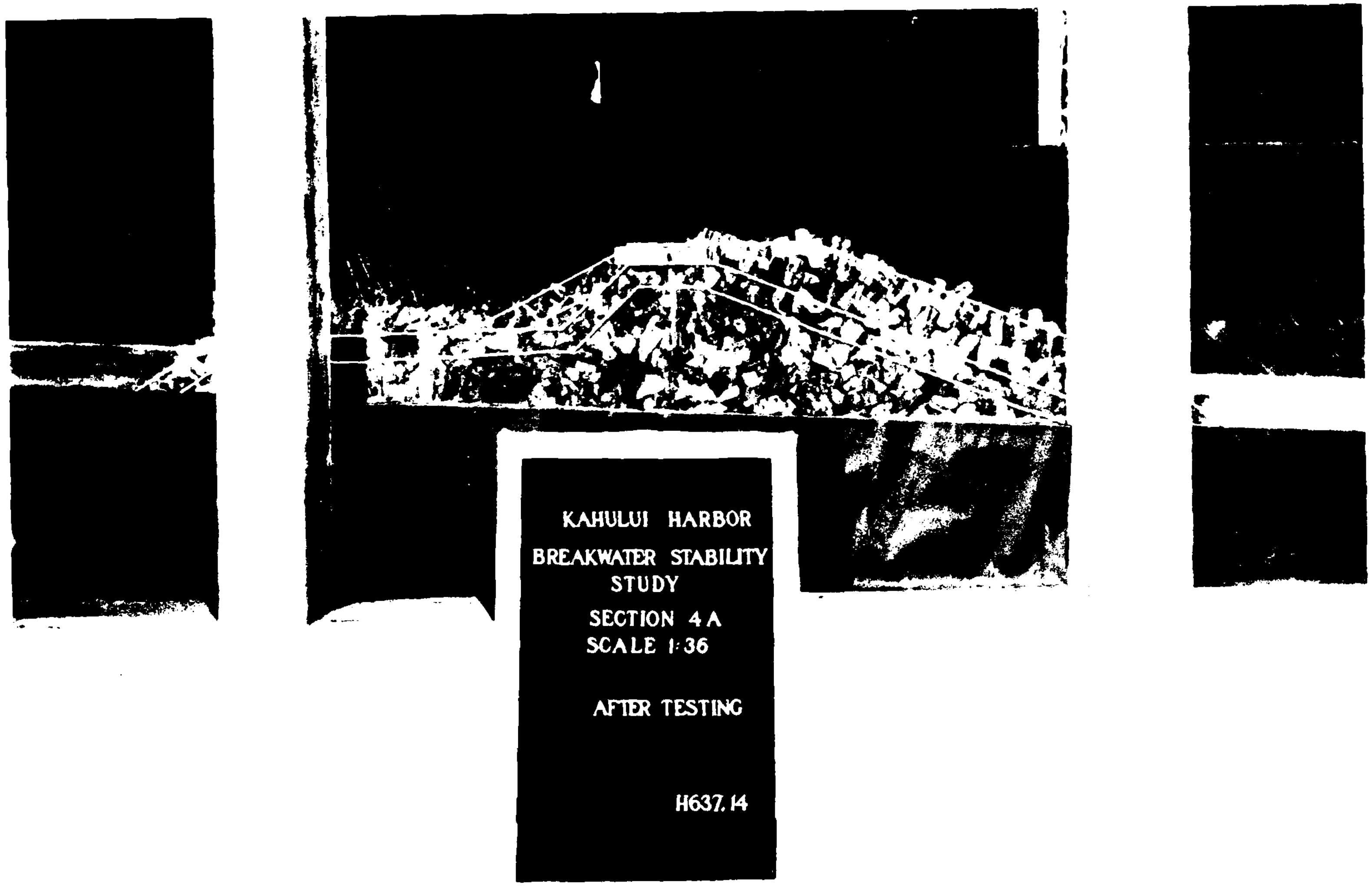


Photo 14. Side view of Plan 1A after testing

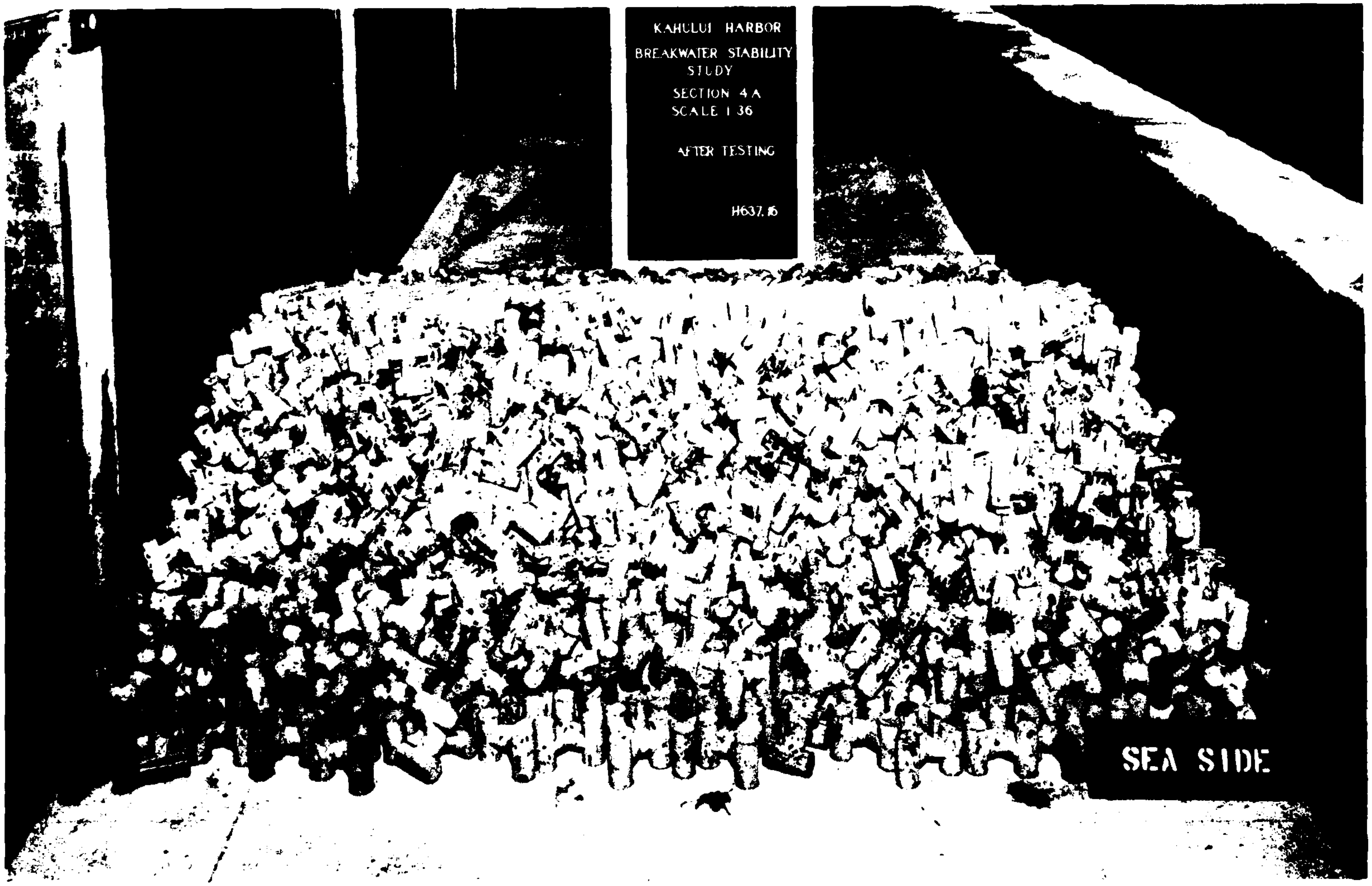


Photo 15. Sea-side view of Plan 1A after testing

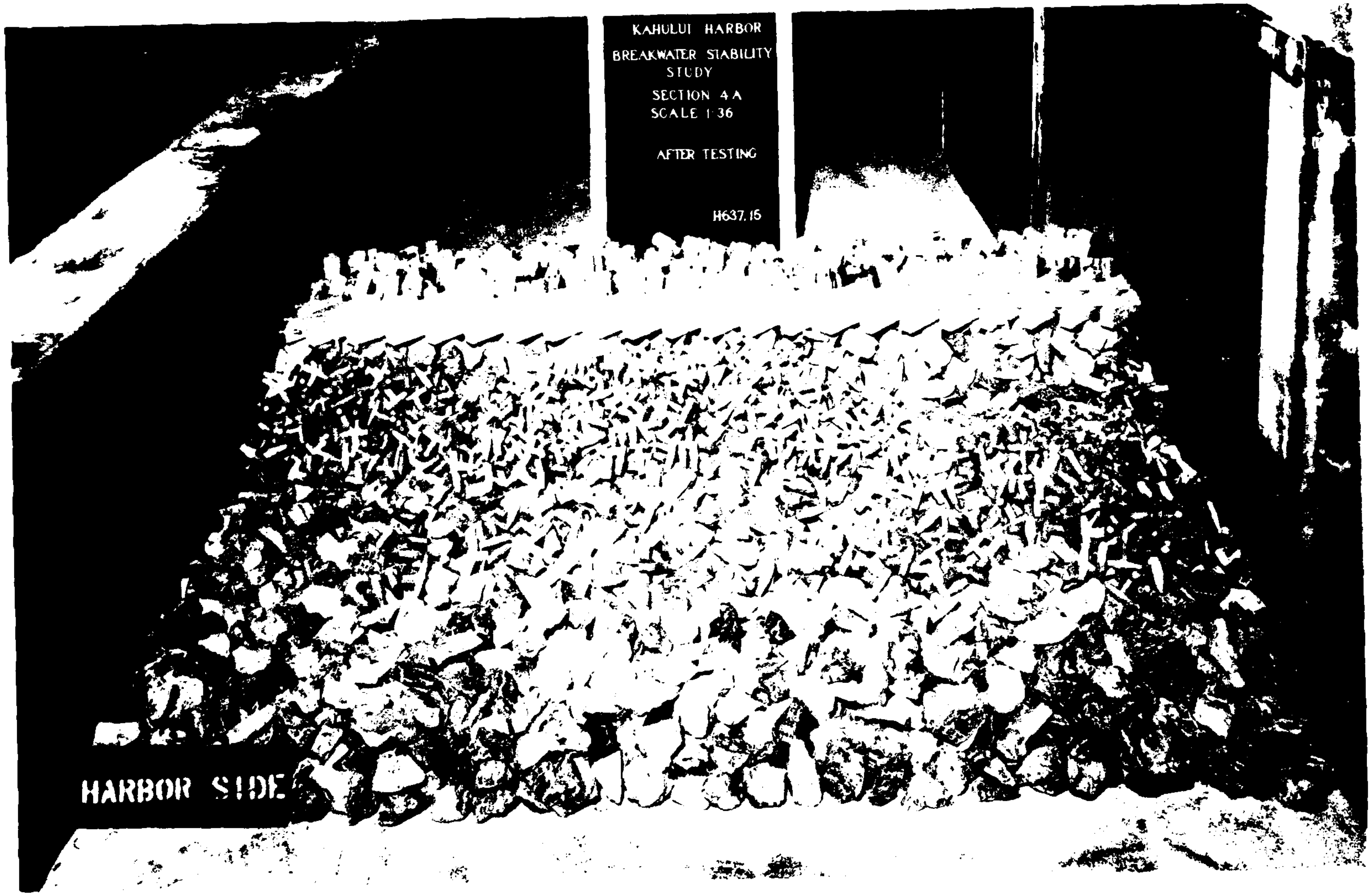


Photo 16. Harbor-side view of Plan 1A after testing

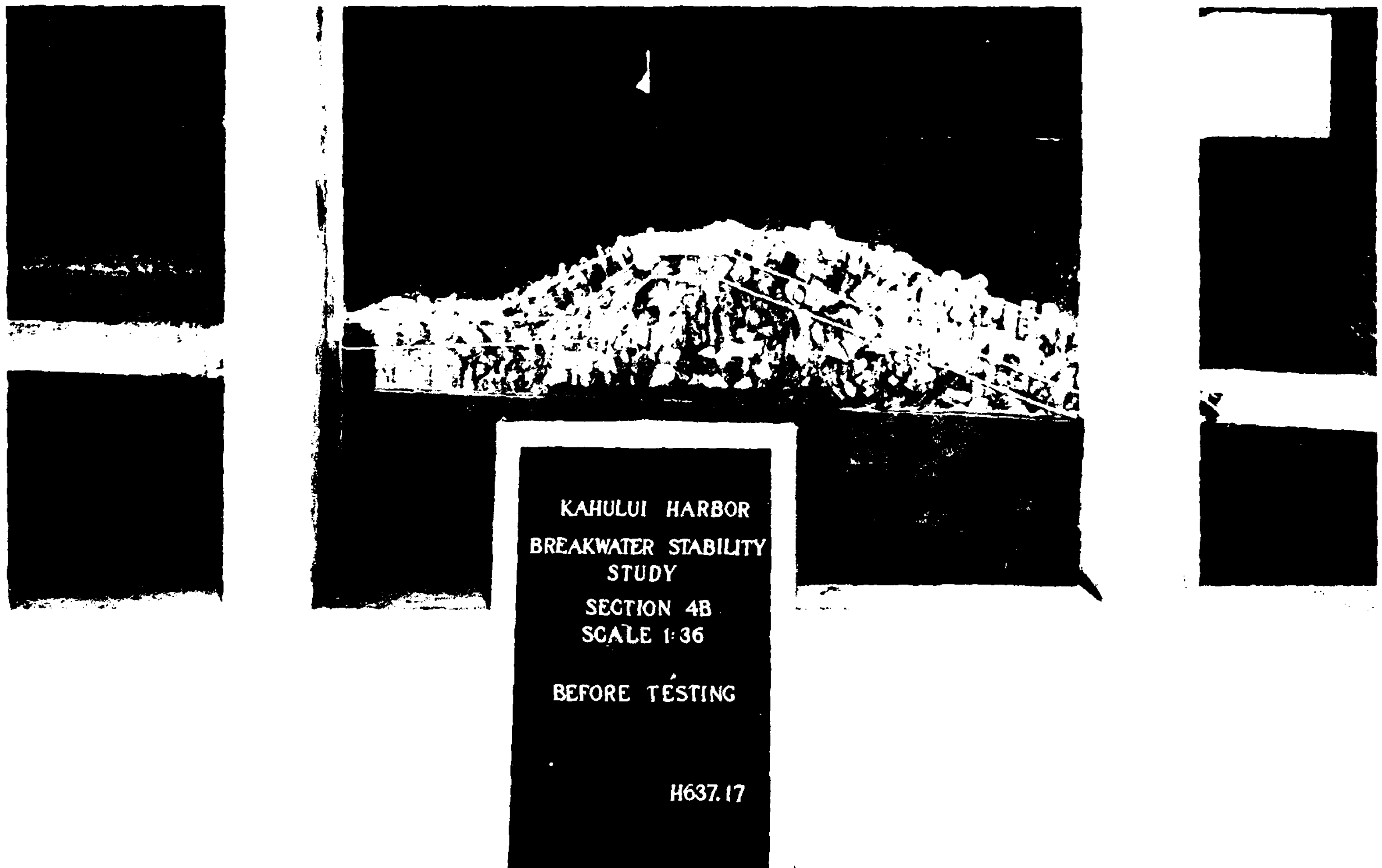


Photo 17. Side view of Plan 1B before testing

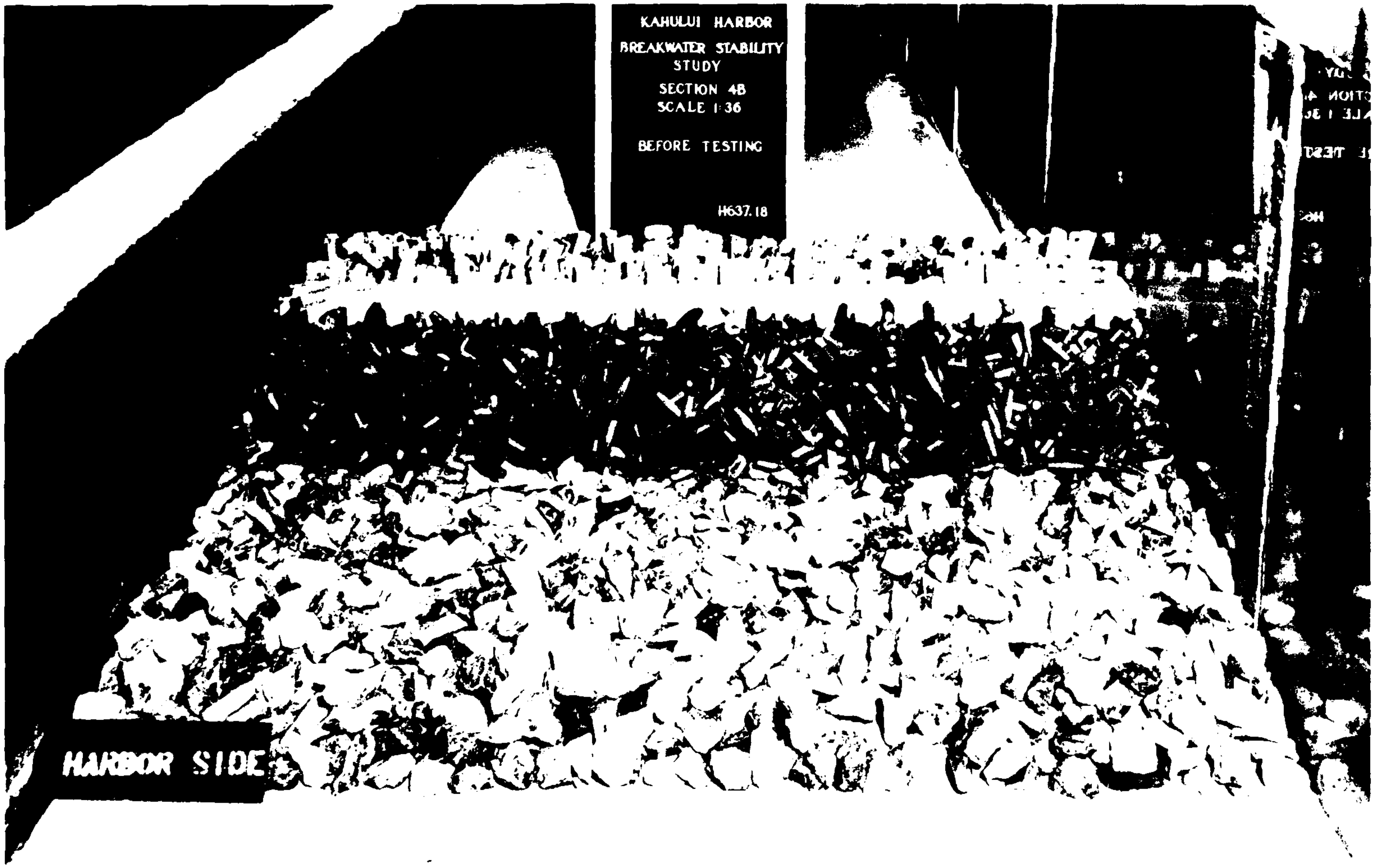


Photo 18. Harbor-side view of Plan 1B before testing

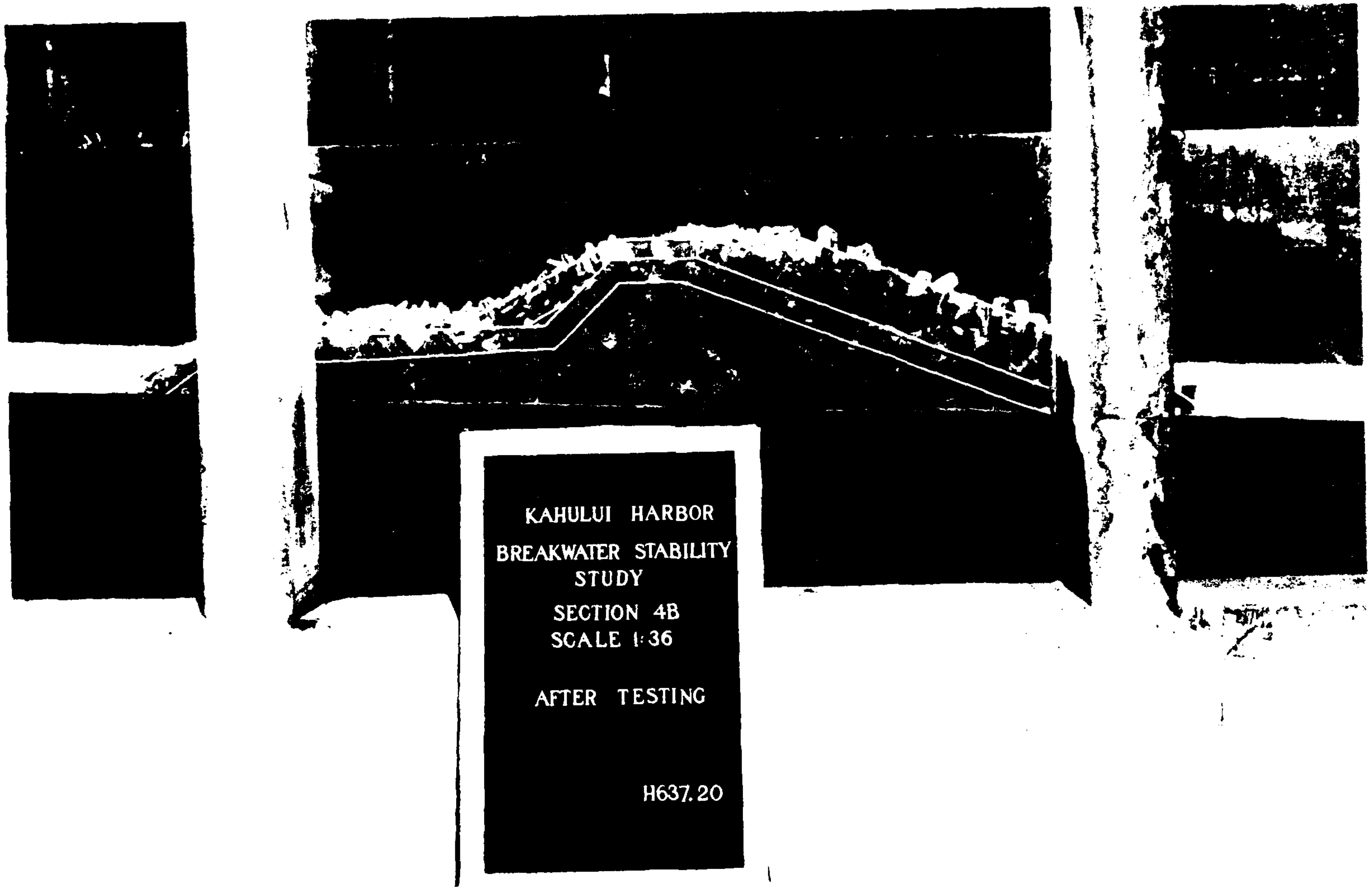


Photo 19. Side view of Plan 1B after testing

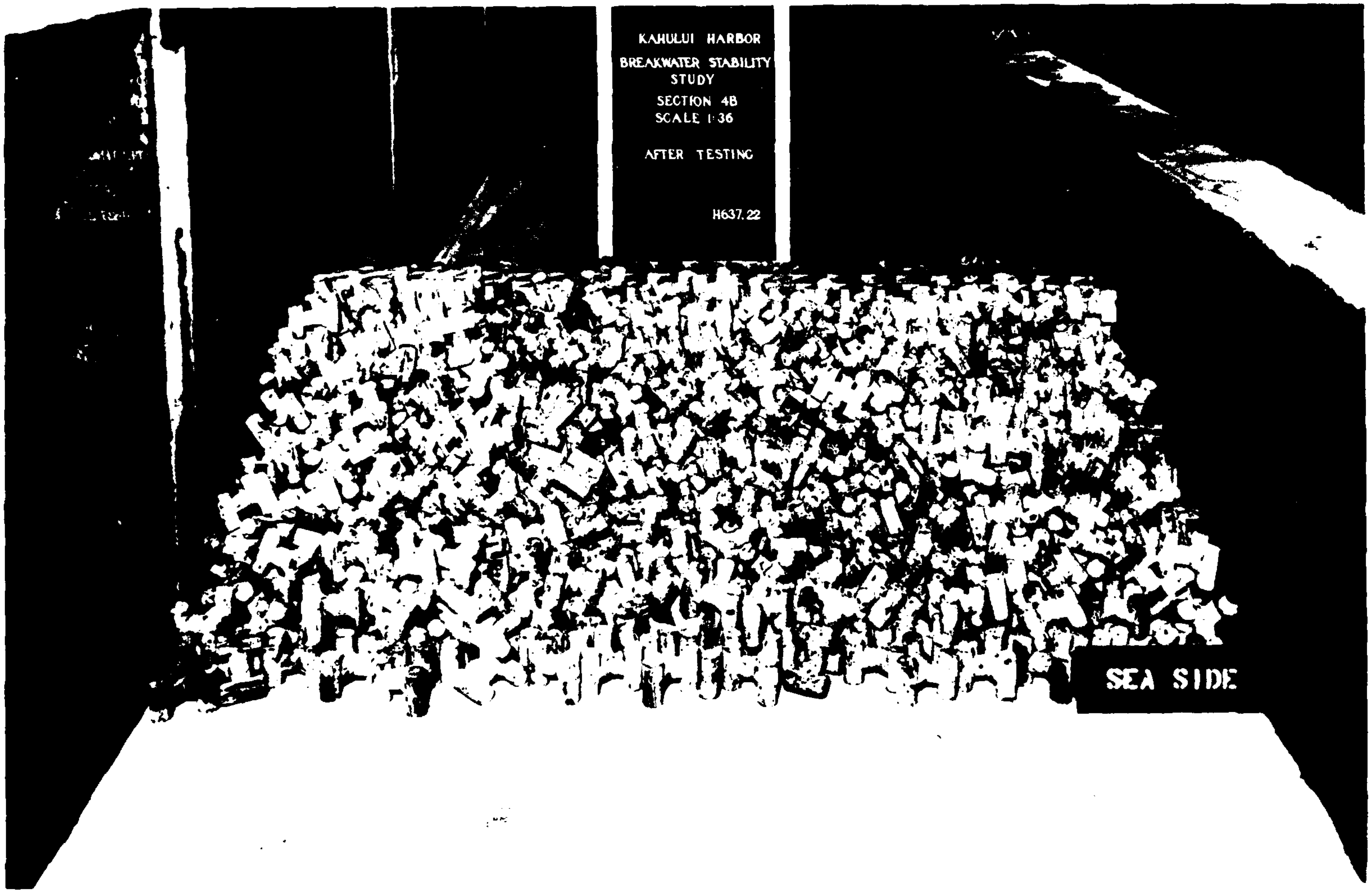


Photo 20. Sea-side view of Plan 1B after testing

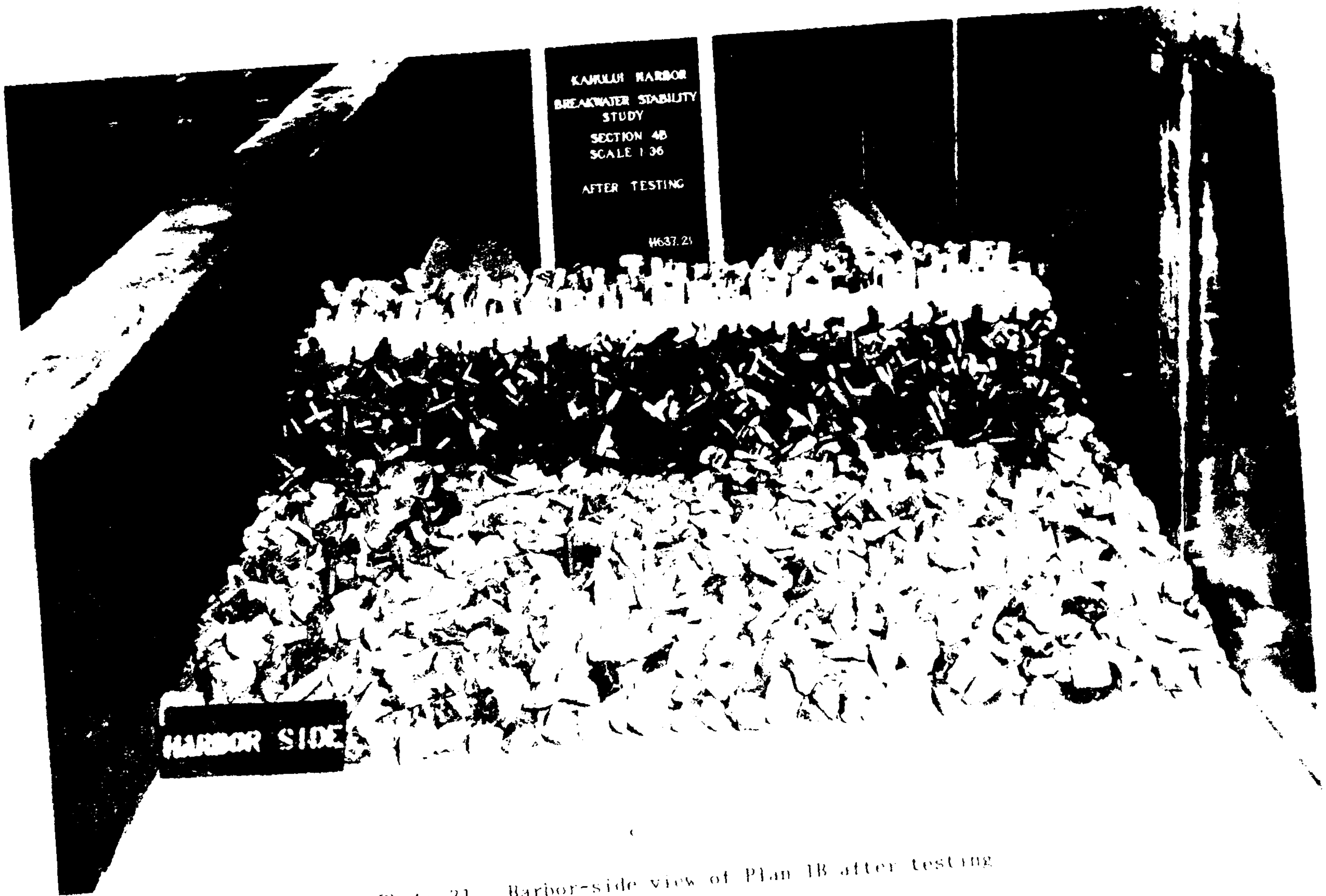


Photo 21. Harbor-side view of Plan 1B after testing

Photo 22. Random and special toe placement

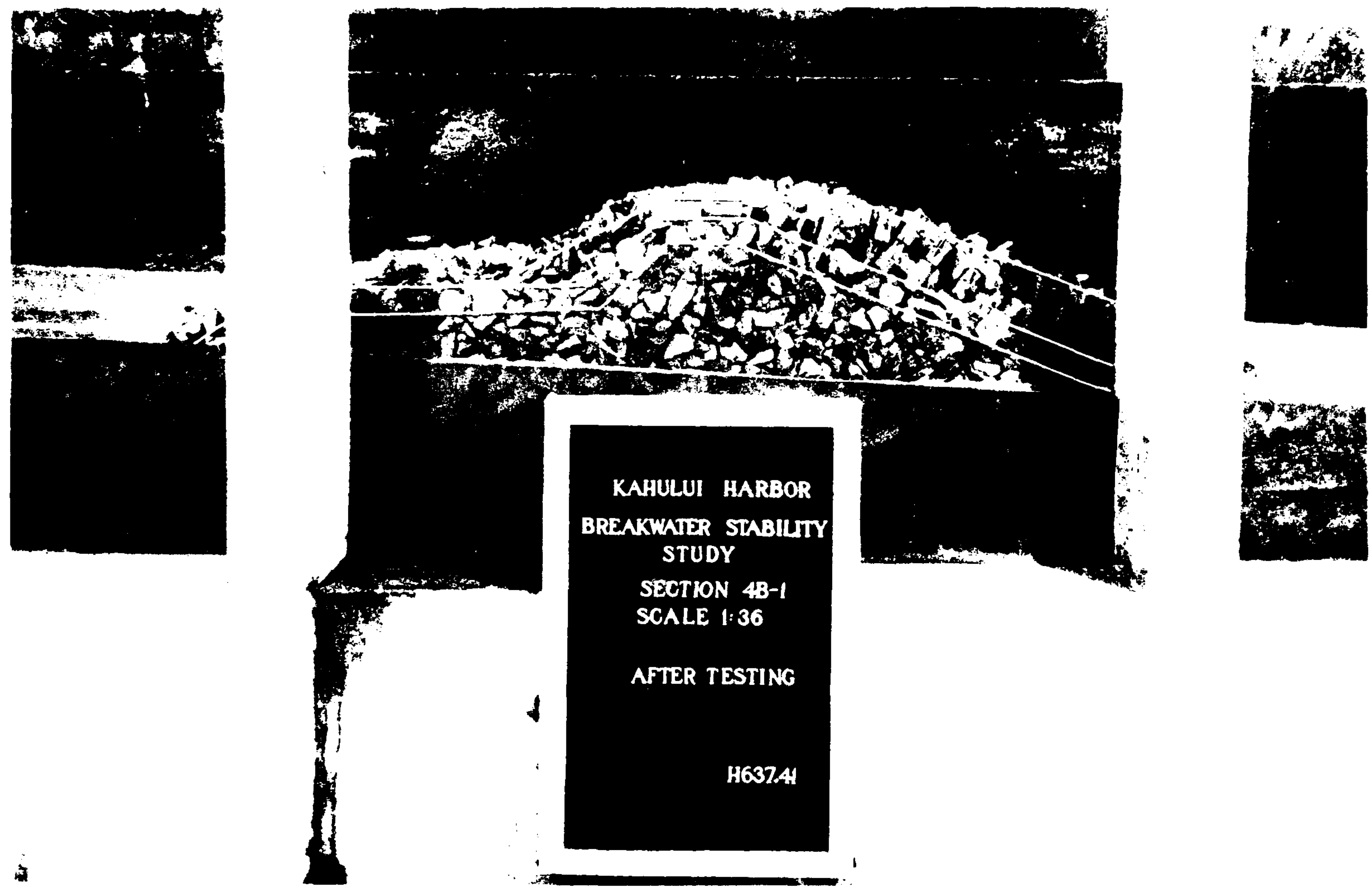


Photo 23. Side view of Plan 1B-1 after testing

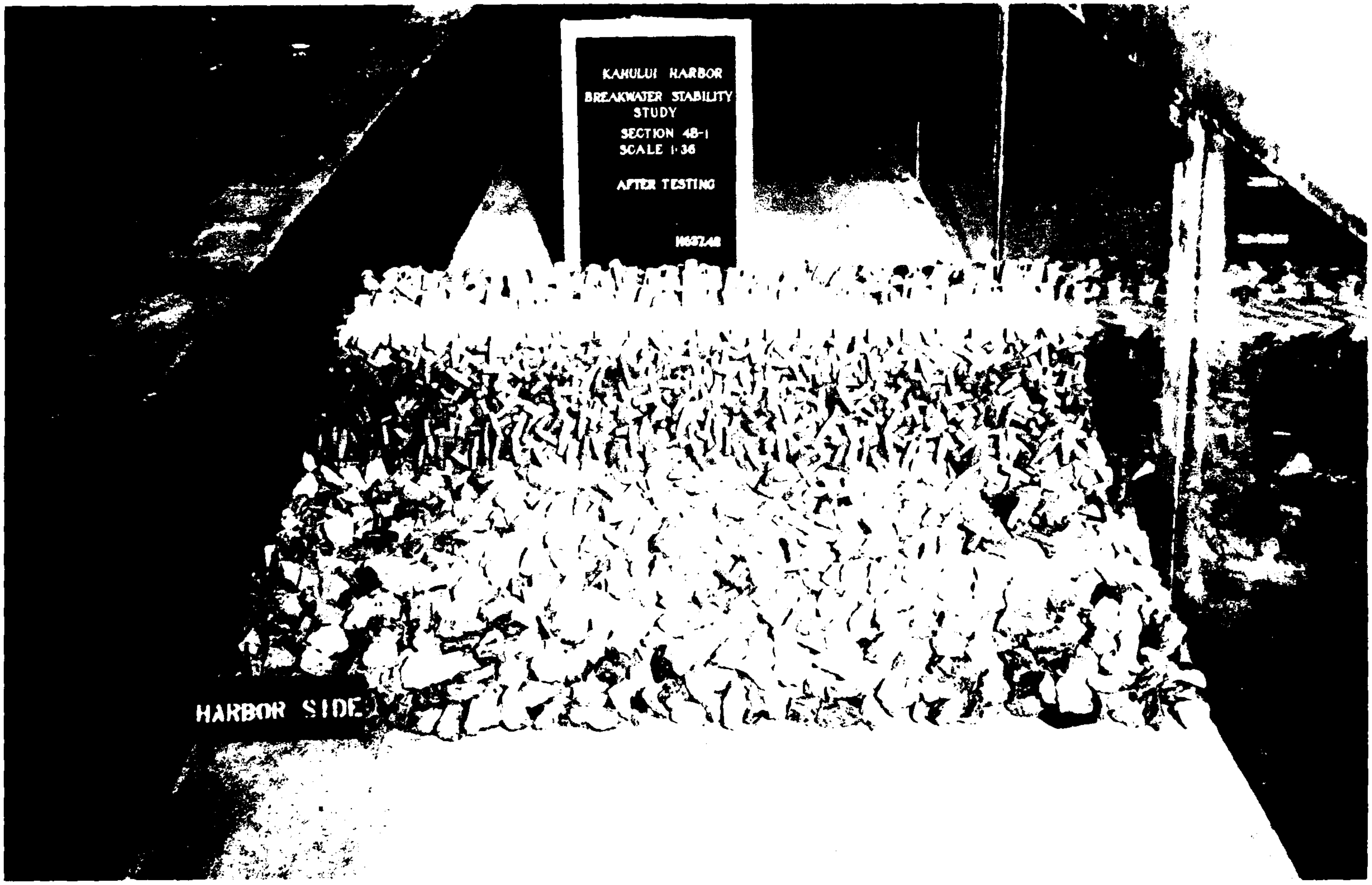


Photo 24. Harbor-side view of Plan 1B-1 after testing

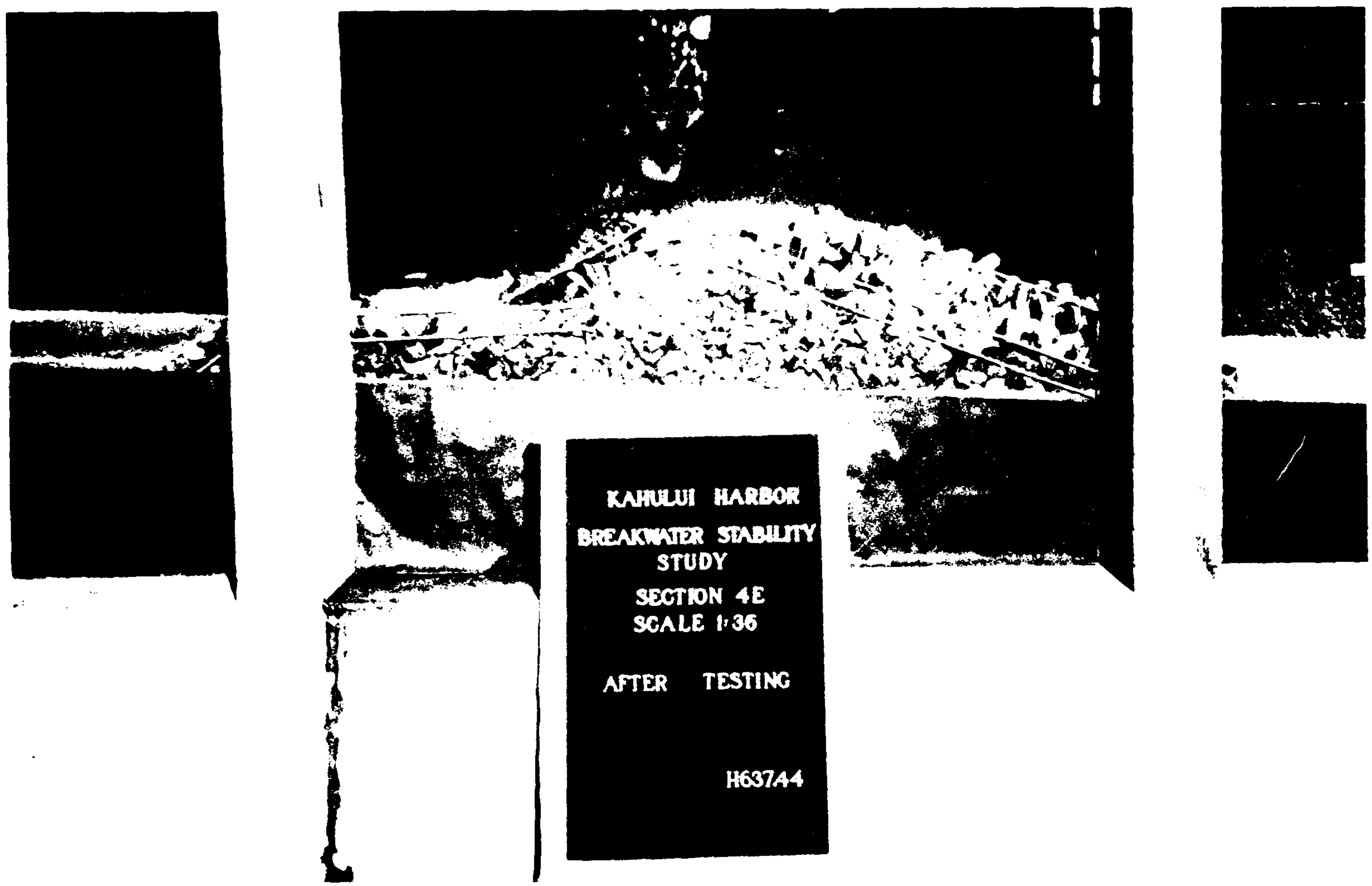


Photo 25. Side view of Plan 1C after testing

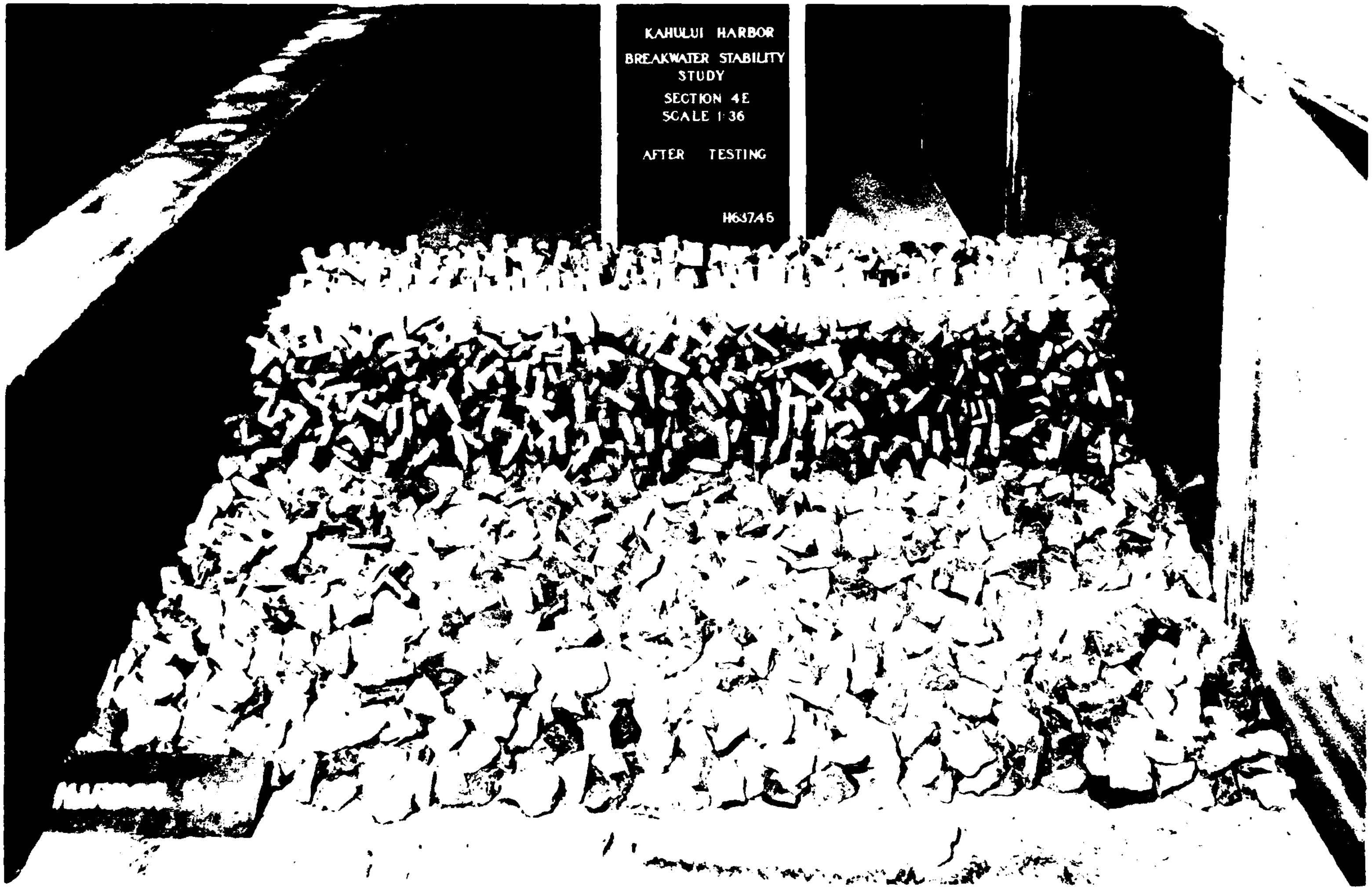


Photo 26. Harbor-side view of Plan 1C after testing

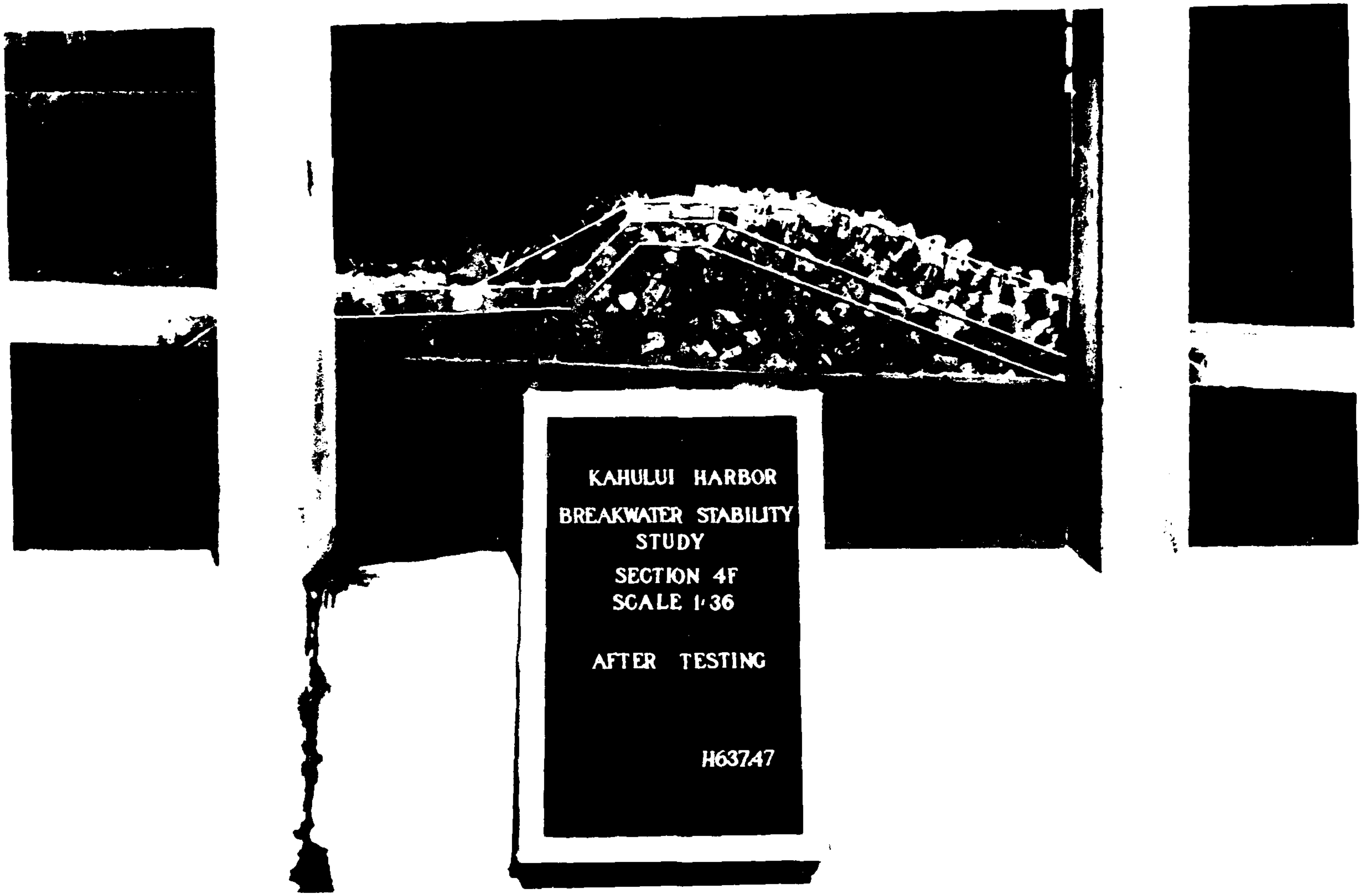


Photo 27. Side view of Plan 1D after testing

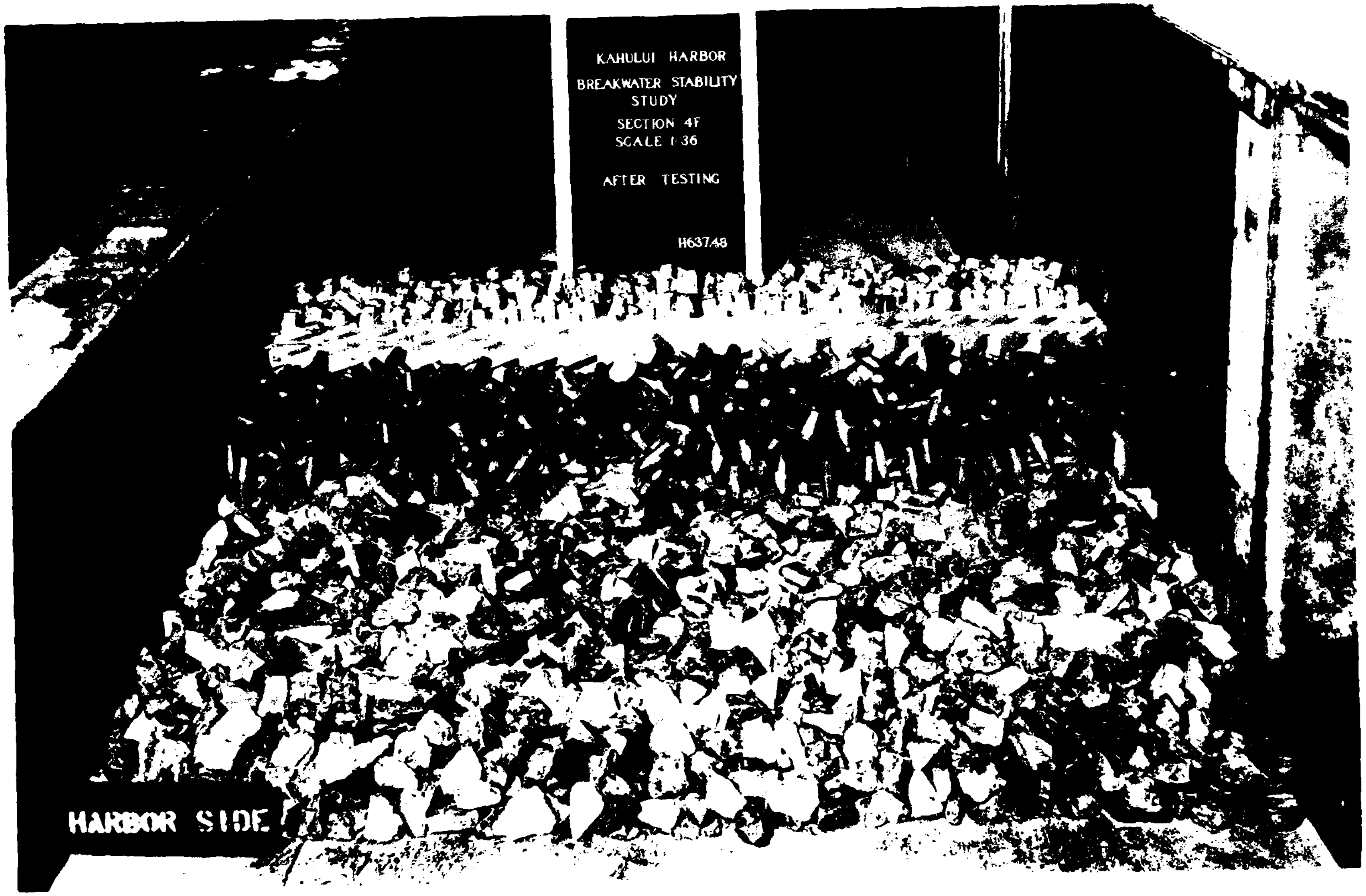


Photo 28. Harbor-side view of Plan 1D after testing

Photo 29. Extra special toe placement

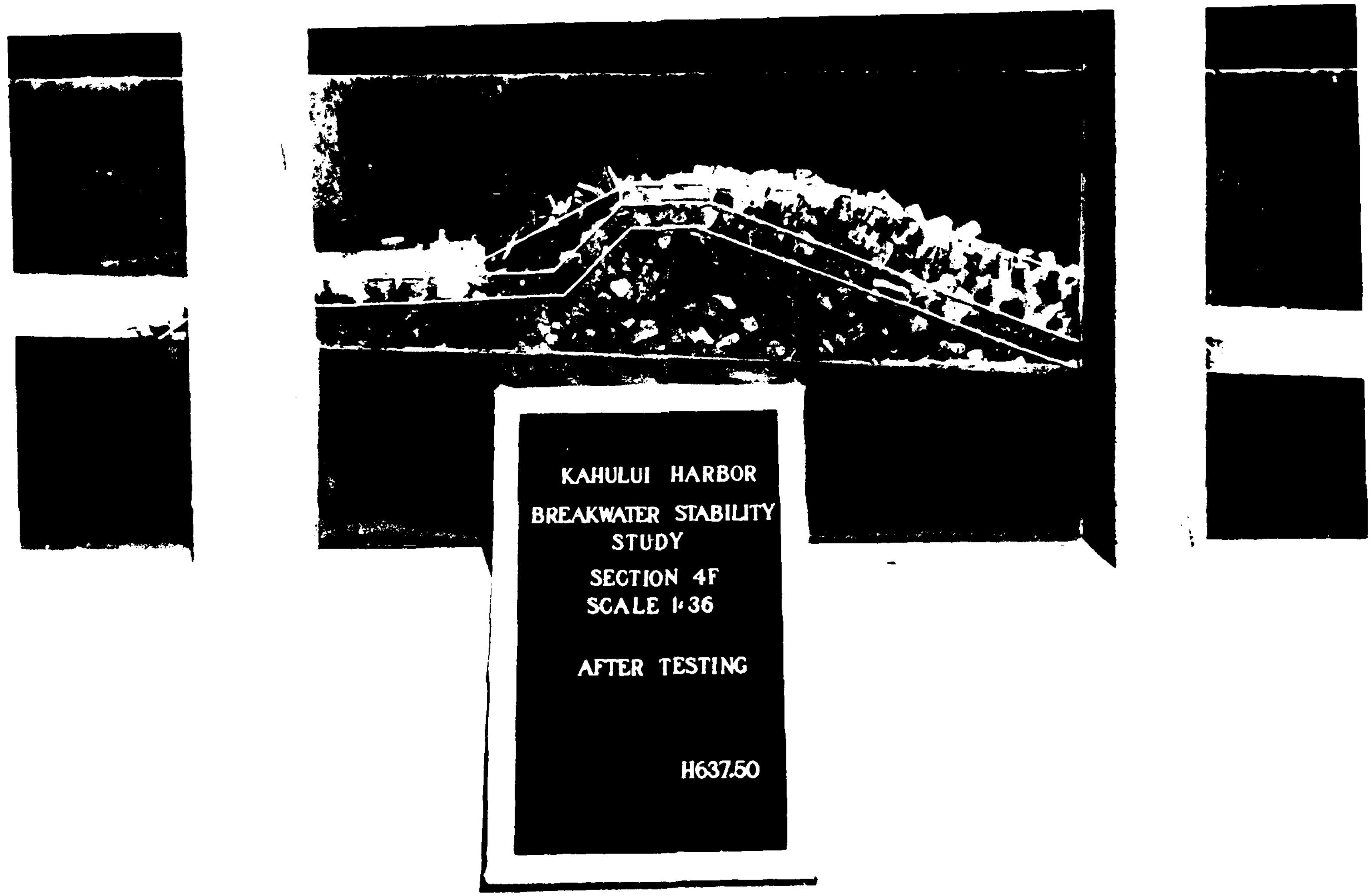


Photo 30. Side view of Plan 1D-1 after testing

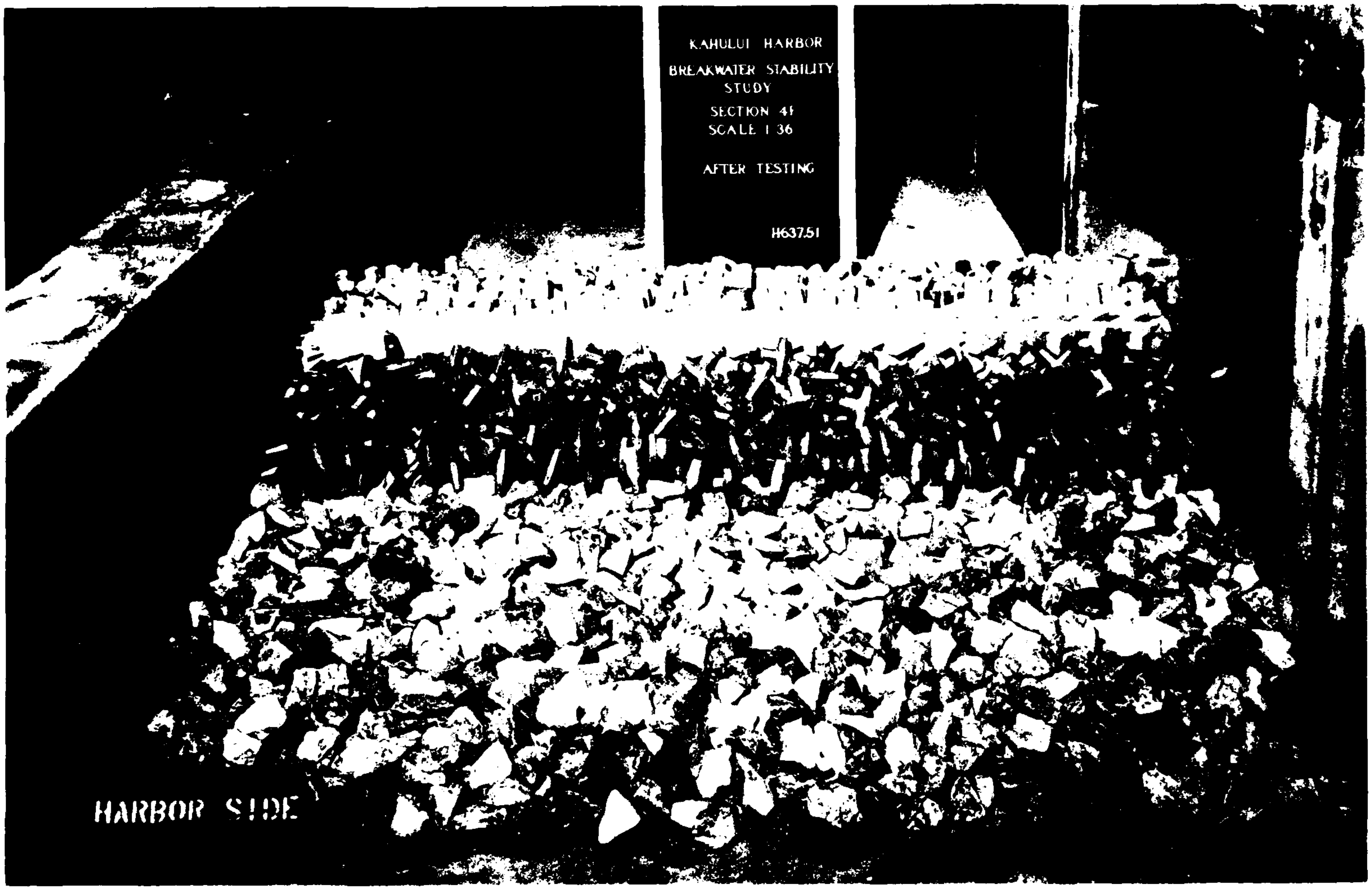


Photo 31. Harbor-side view of Plan 1D-1 after testing

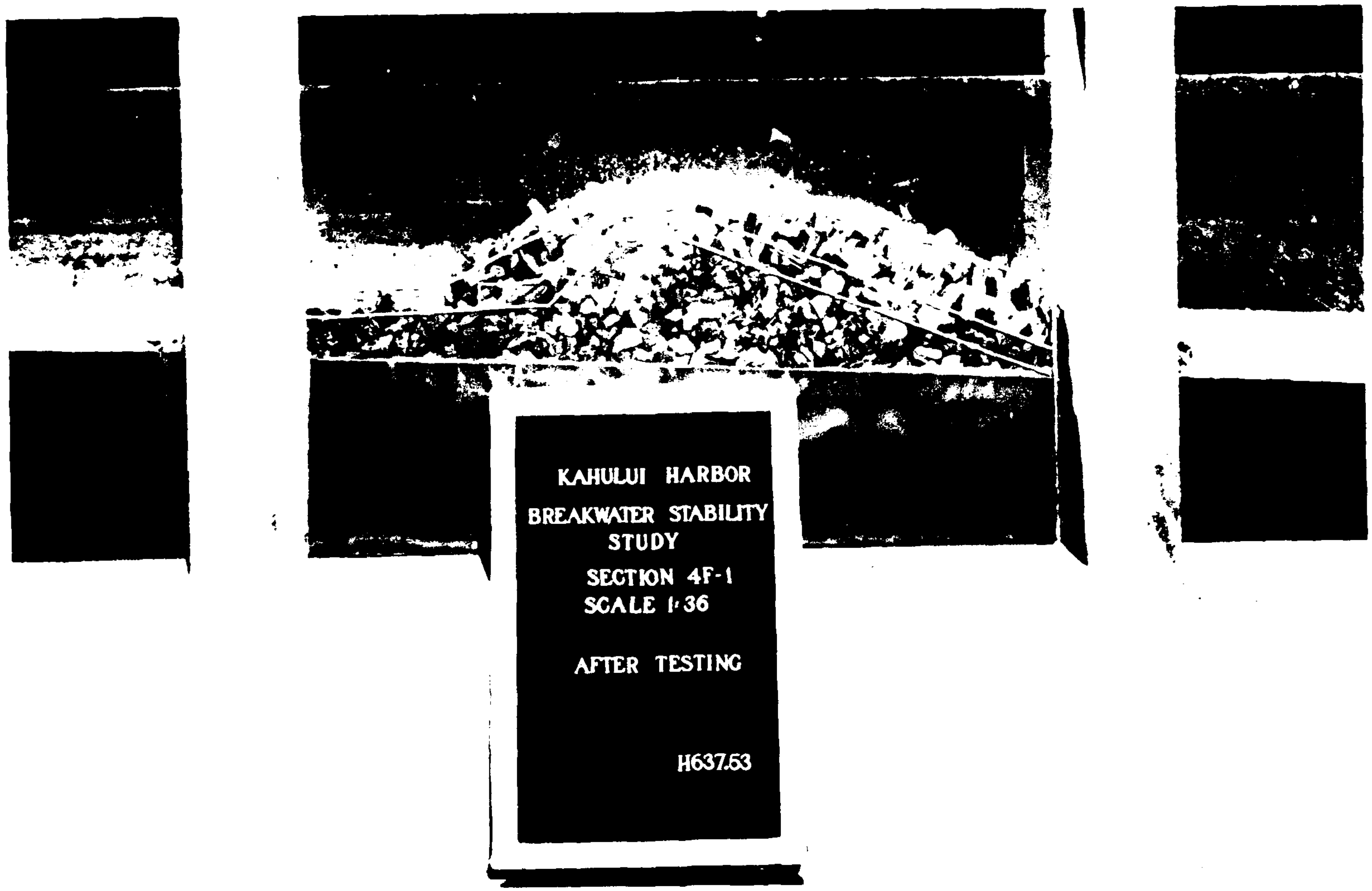


Photo 32. Side view of Plan 1D-2 after testing, 1st test

Photo 33. Harbor-side view of Plan 1D-2 after testing, 1st test

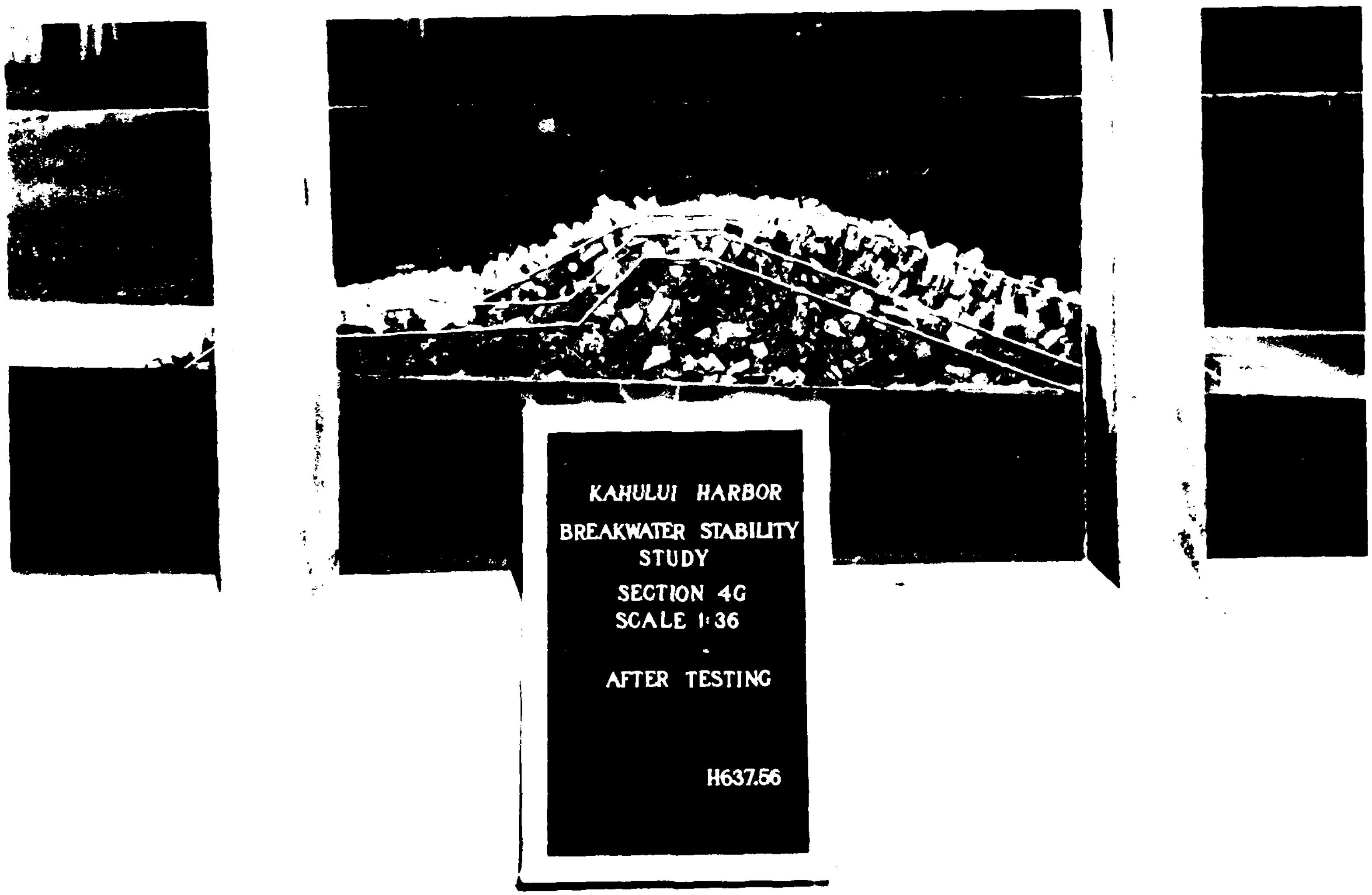


Photo 34. Side view of Plan 1E after testing

Photo 35. Harbor-side view of Plan 1E after testing

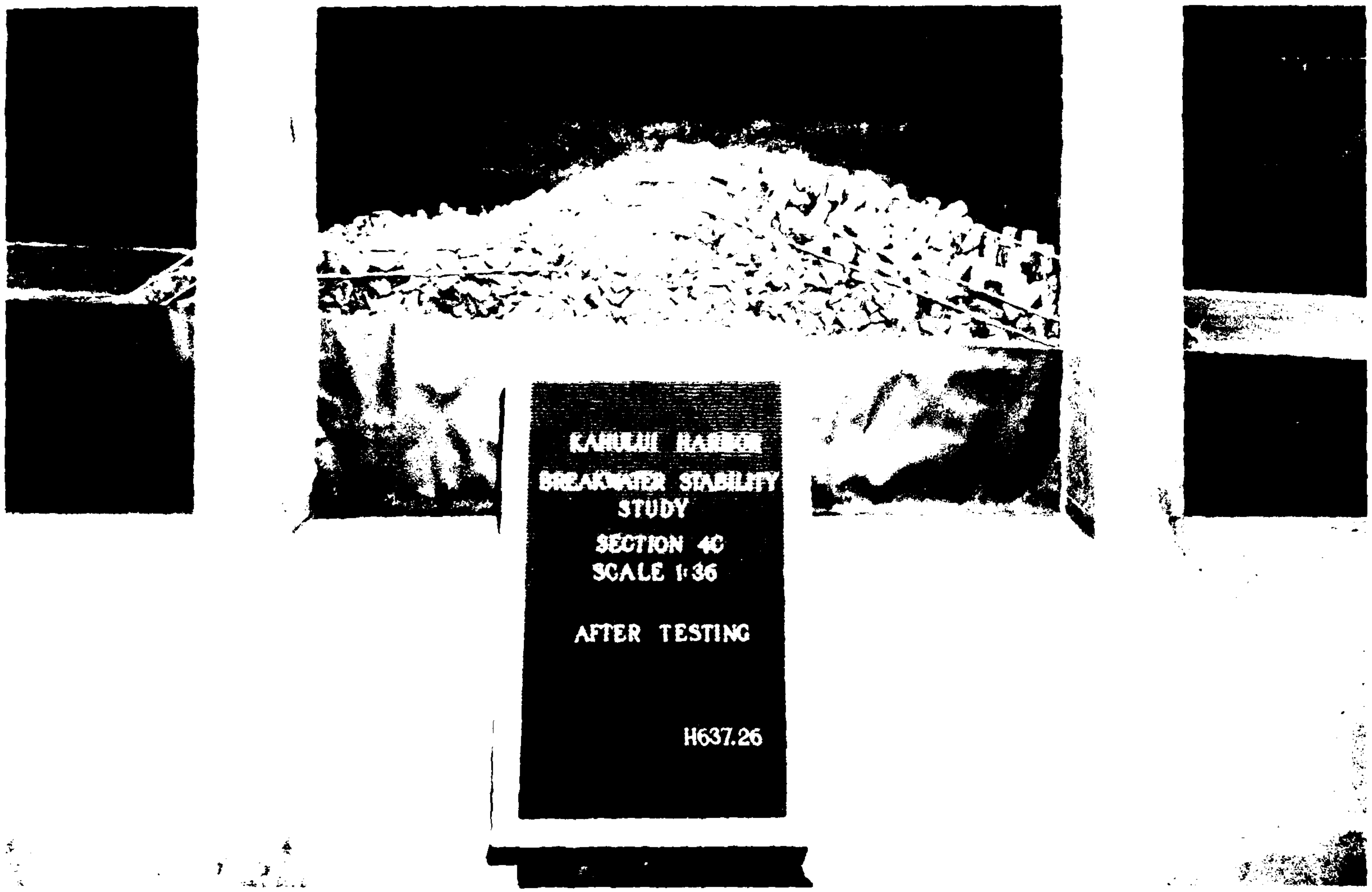


Photo 36. Side view of Plan 1F after testing

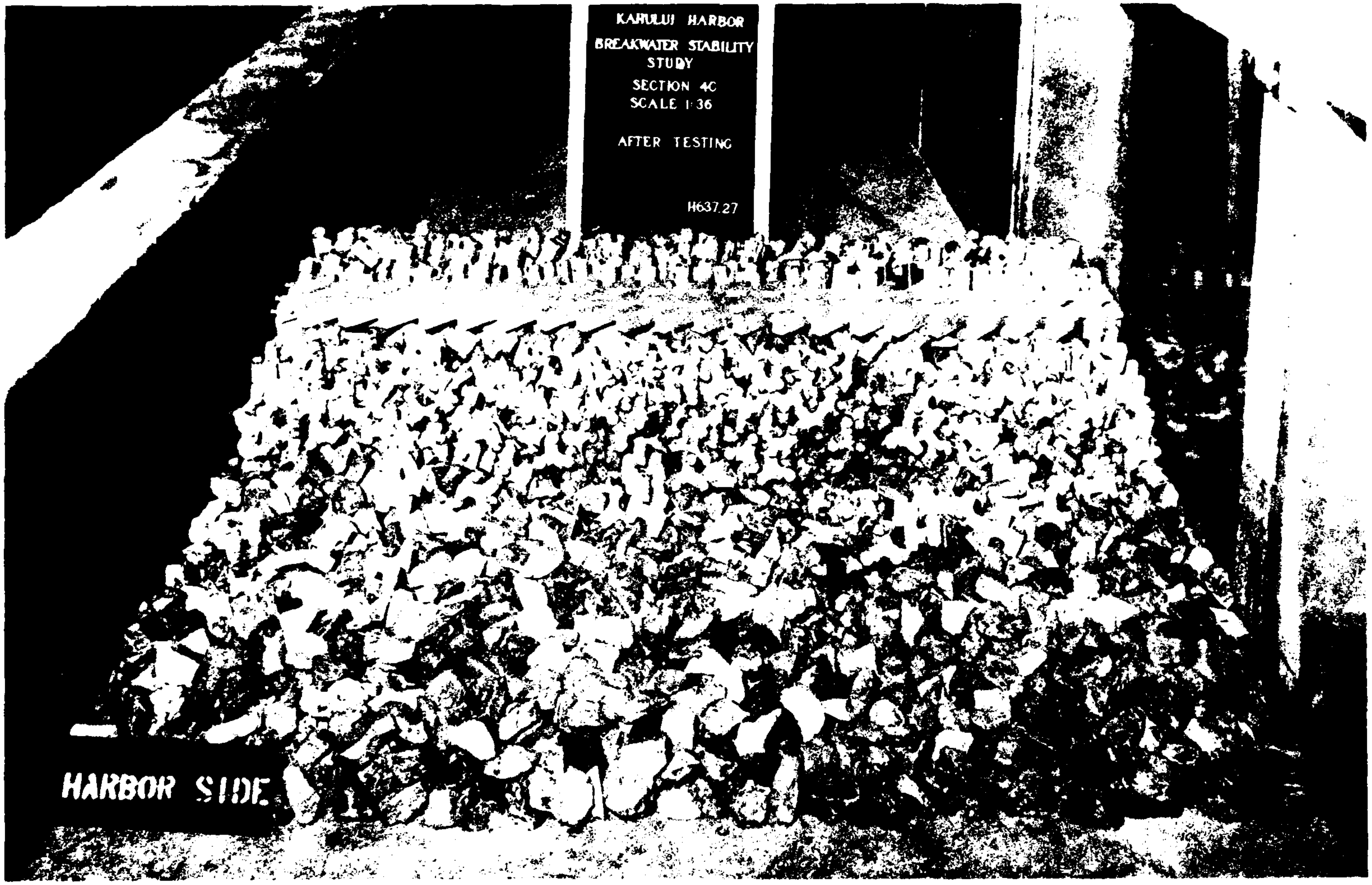


Photo 37. Harbor-side view of Plan 1F after testing

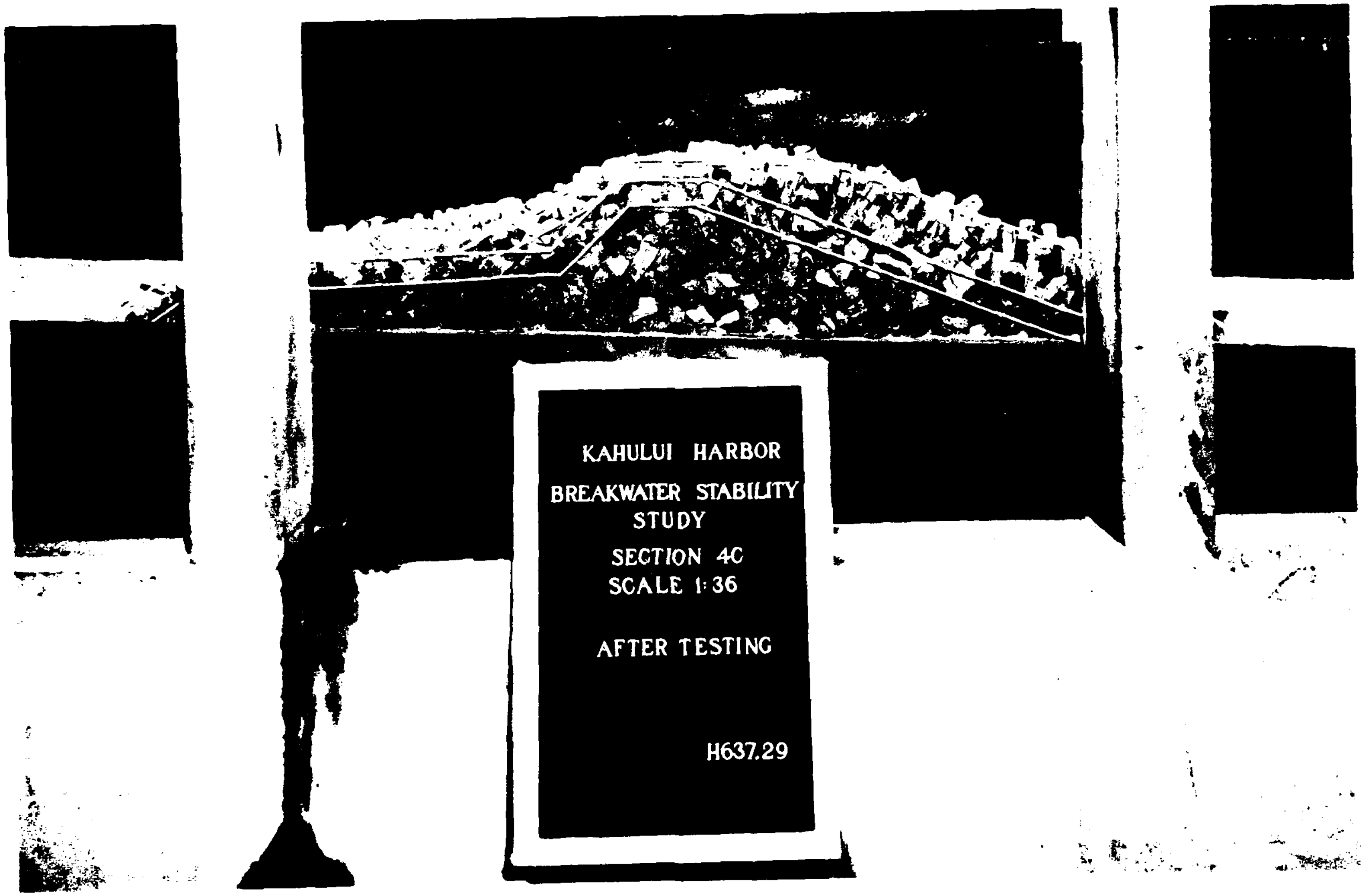


Photo 38. Side view of Plan 1F after testing, 2nd test

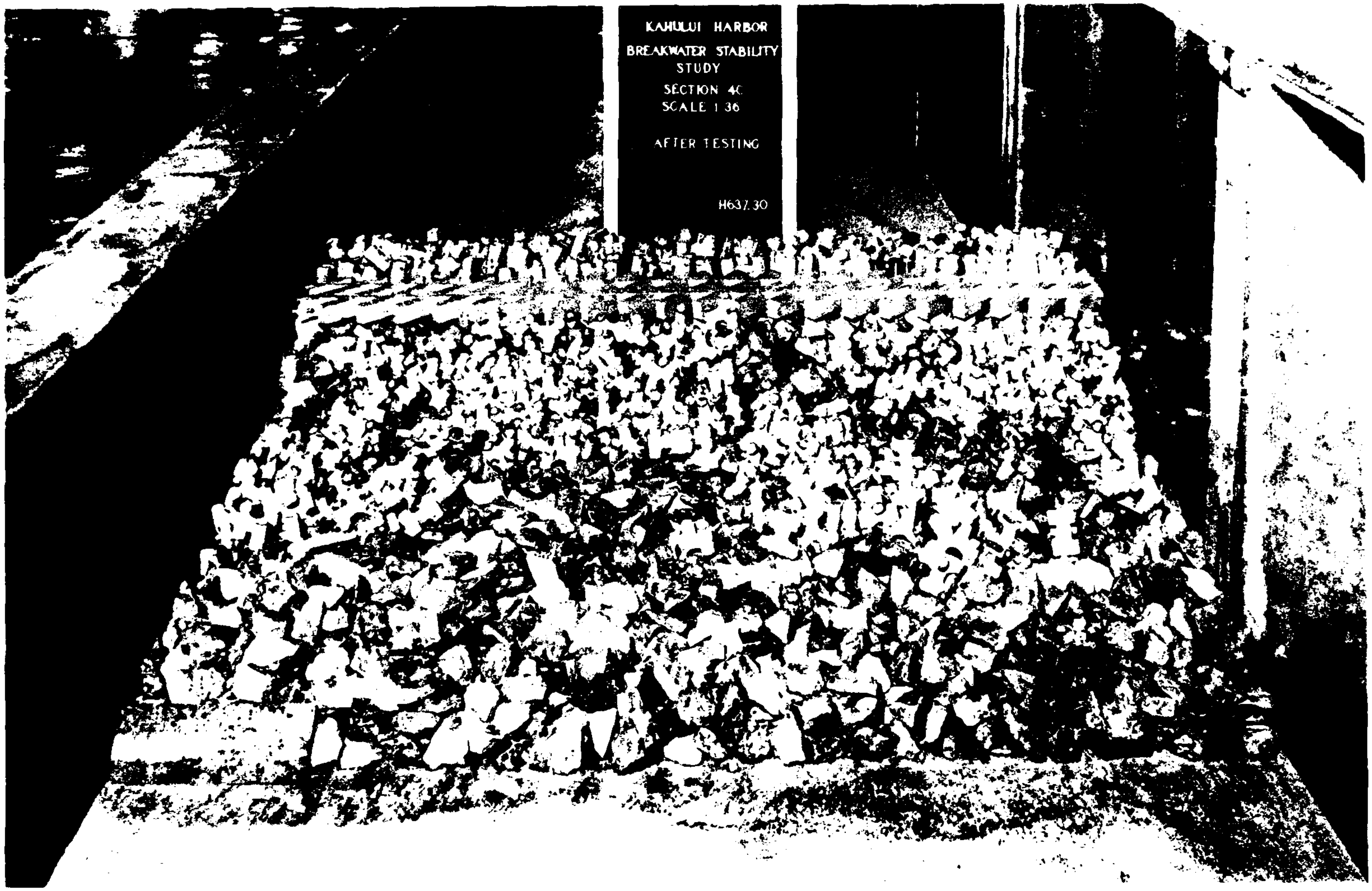


Photo 39. Harbor-side view of Plan 1F after testing, 2nd test

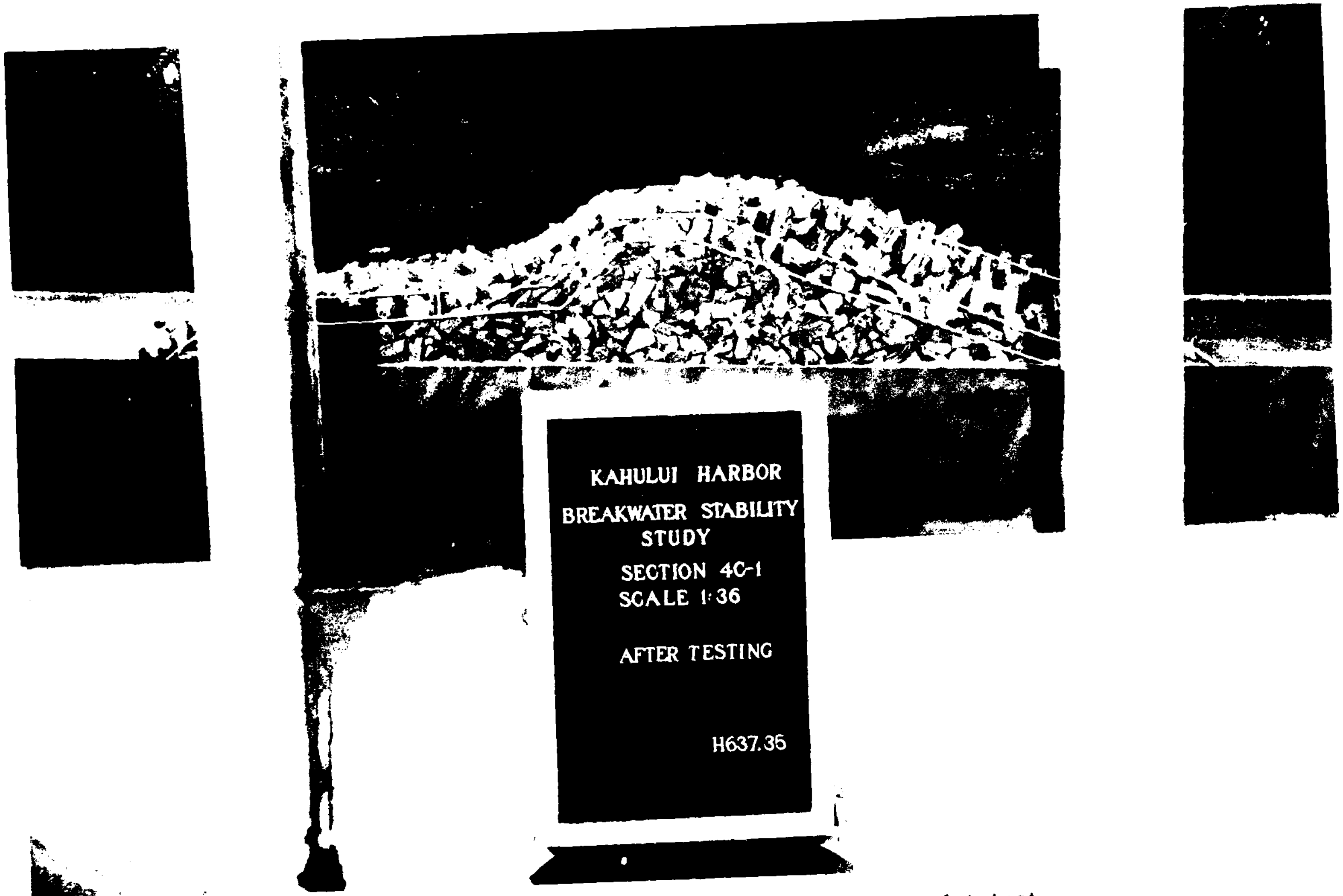


Photo 40. Side view of Plan 1F-1 after testing, 1st test

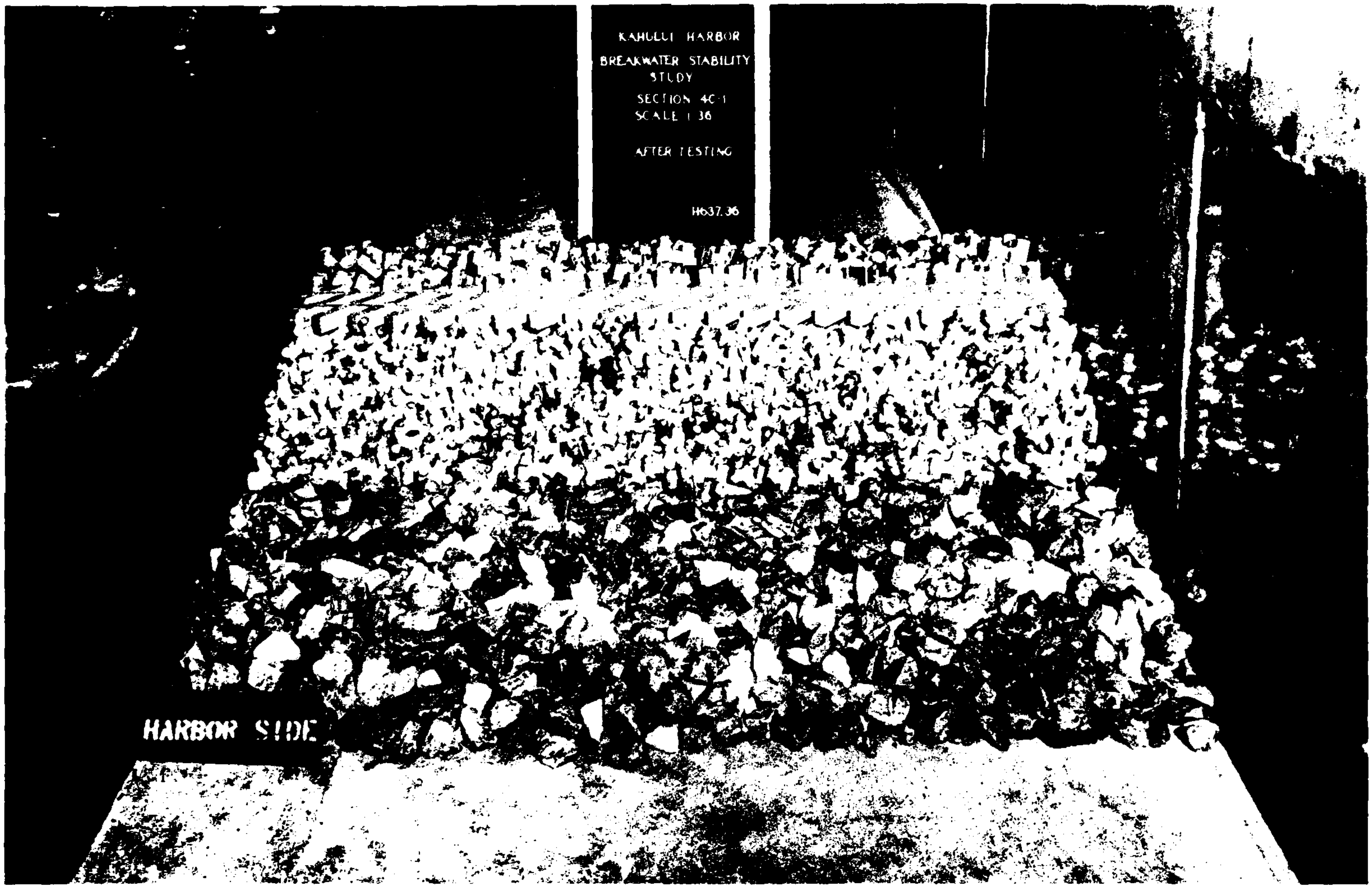


Photo 41. Harbor-side view of Plan 1F-1 after testing, 1st test

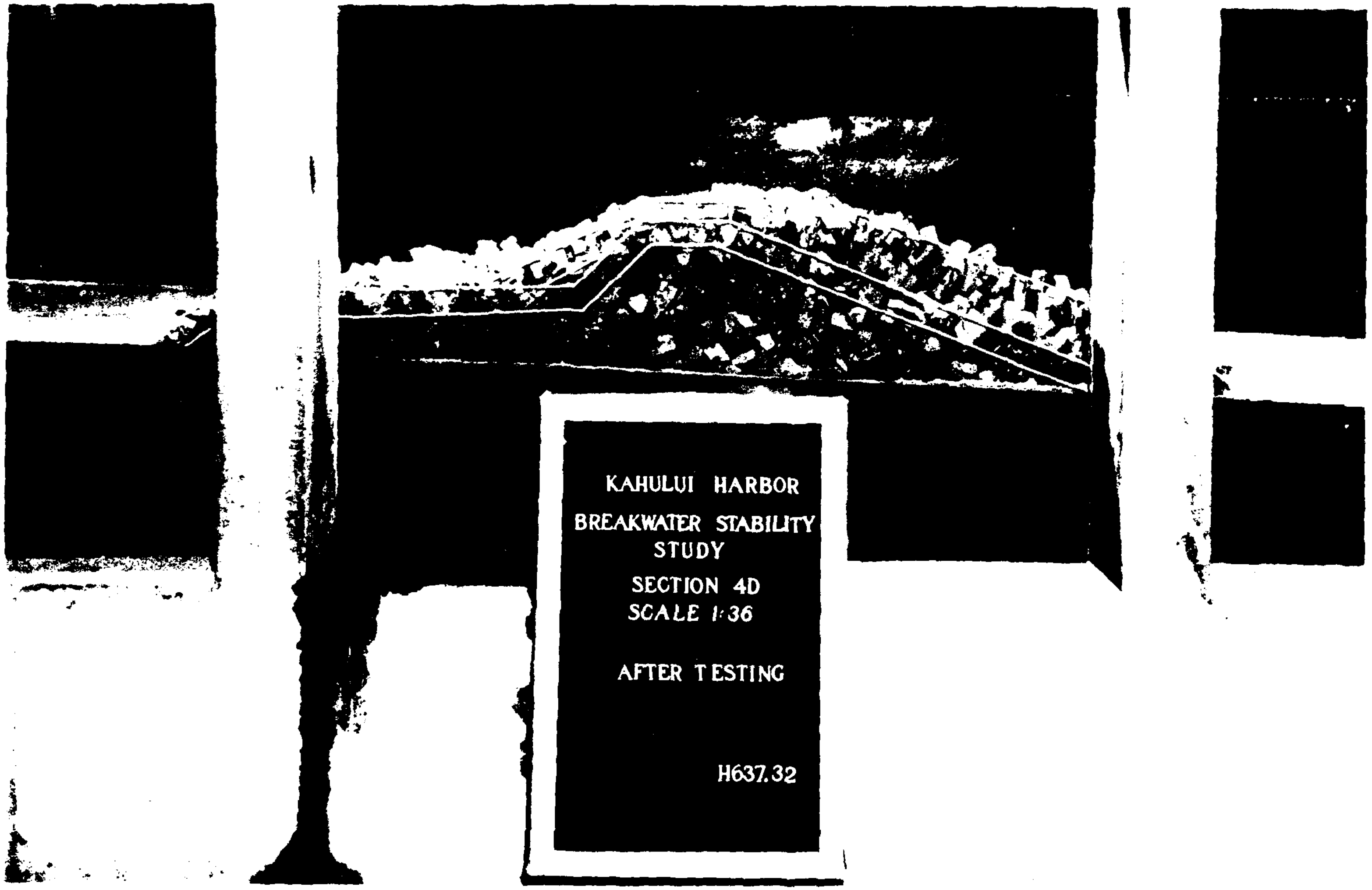


Photo 42. Side view of Plan 1G after testing

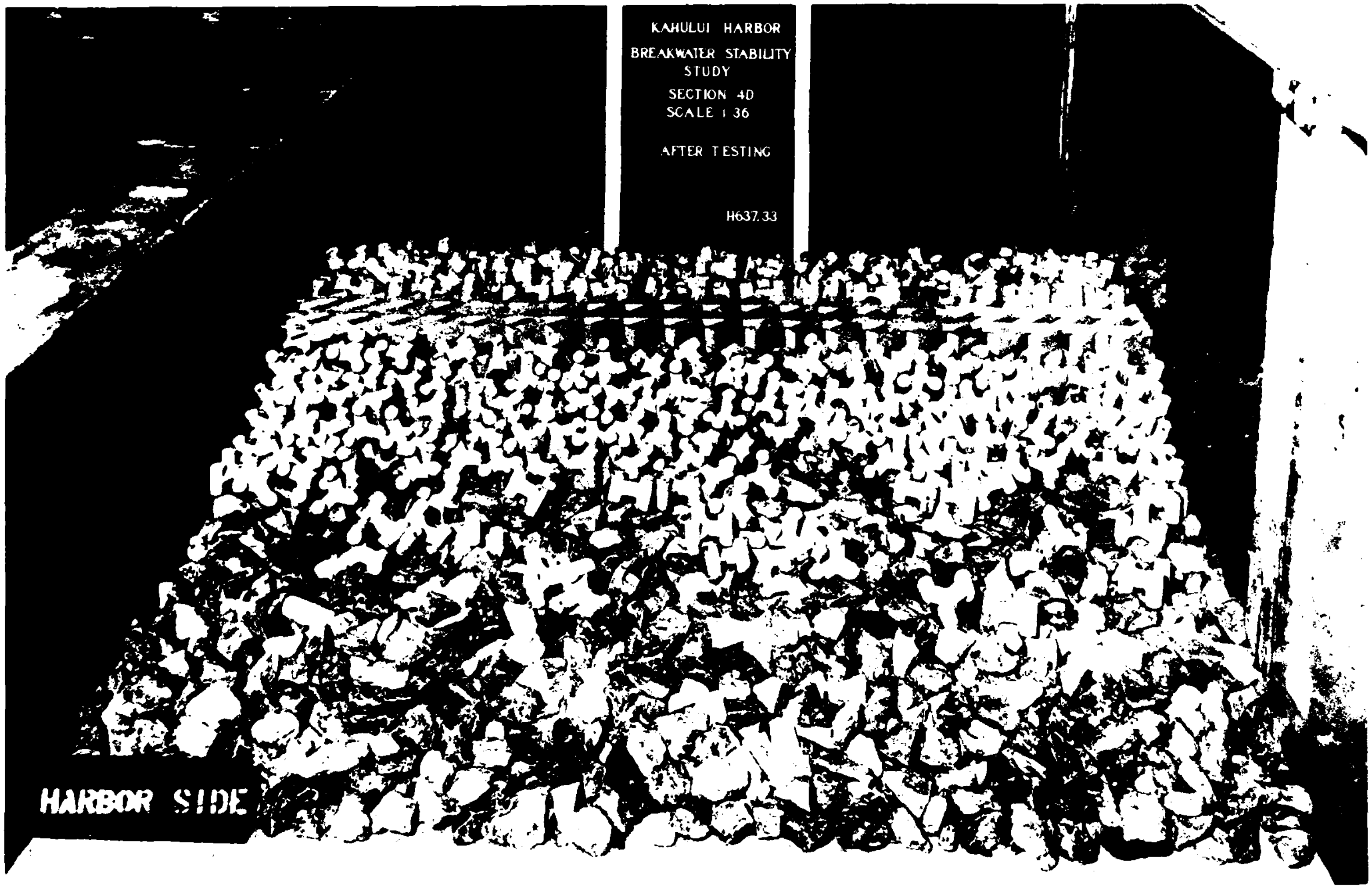


Photo 43. Harbor-side view of Plan 1G after testing

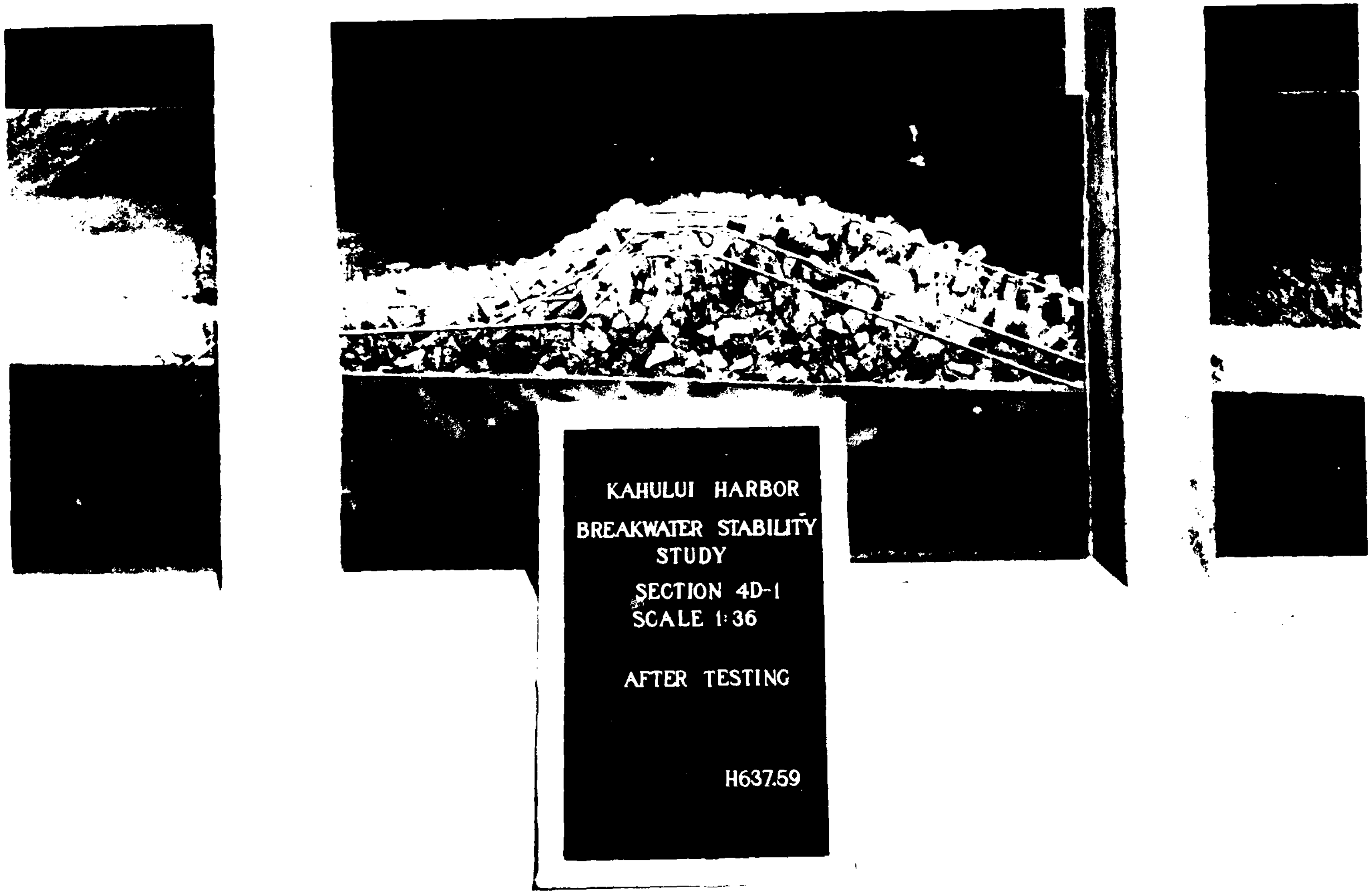


Photo 44. Side view of Plan 1G-1 after testing

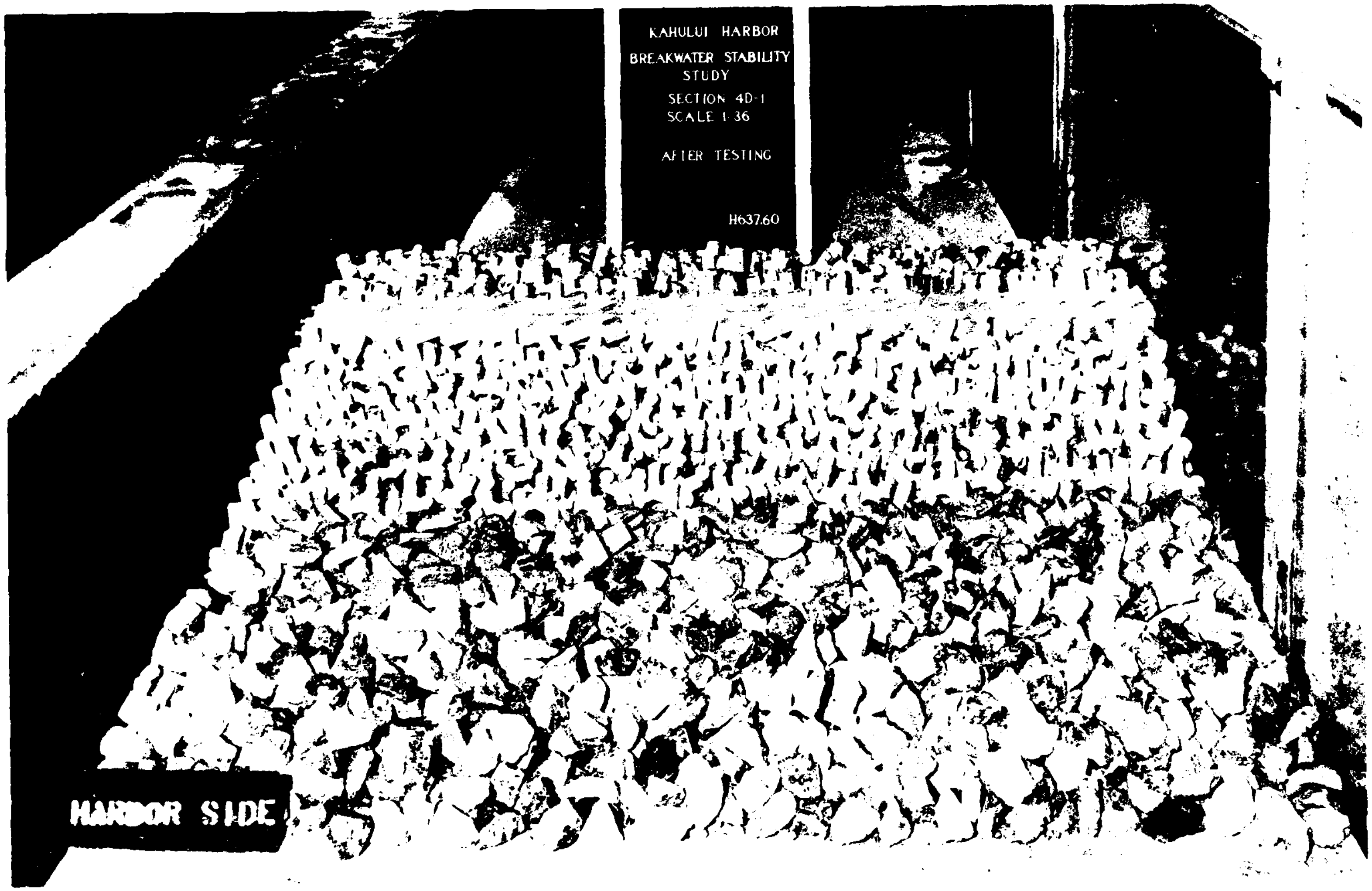


Photo 45. Harbor-side view of Plan 1G-1 after testing

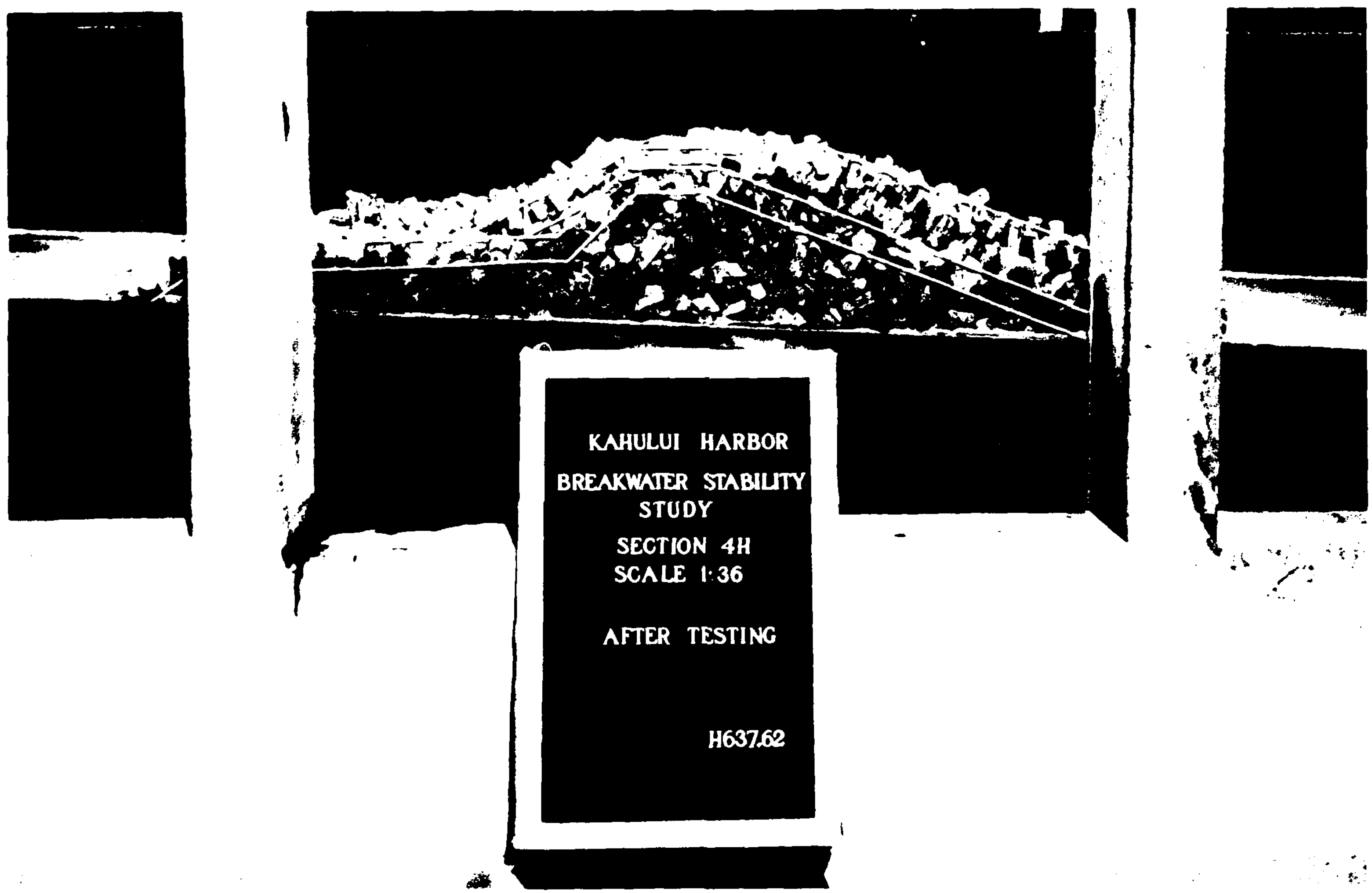


Photo 46. Side view of Plan 1H after testing

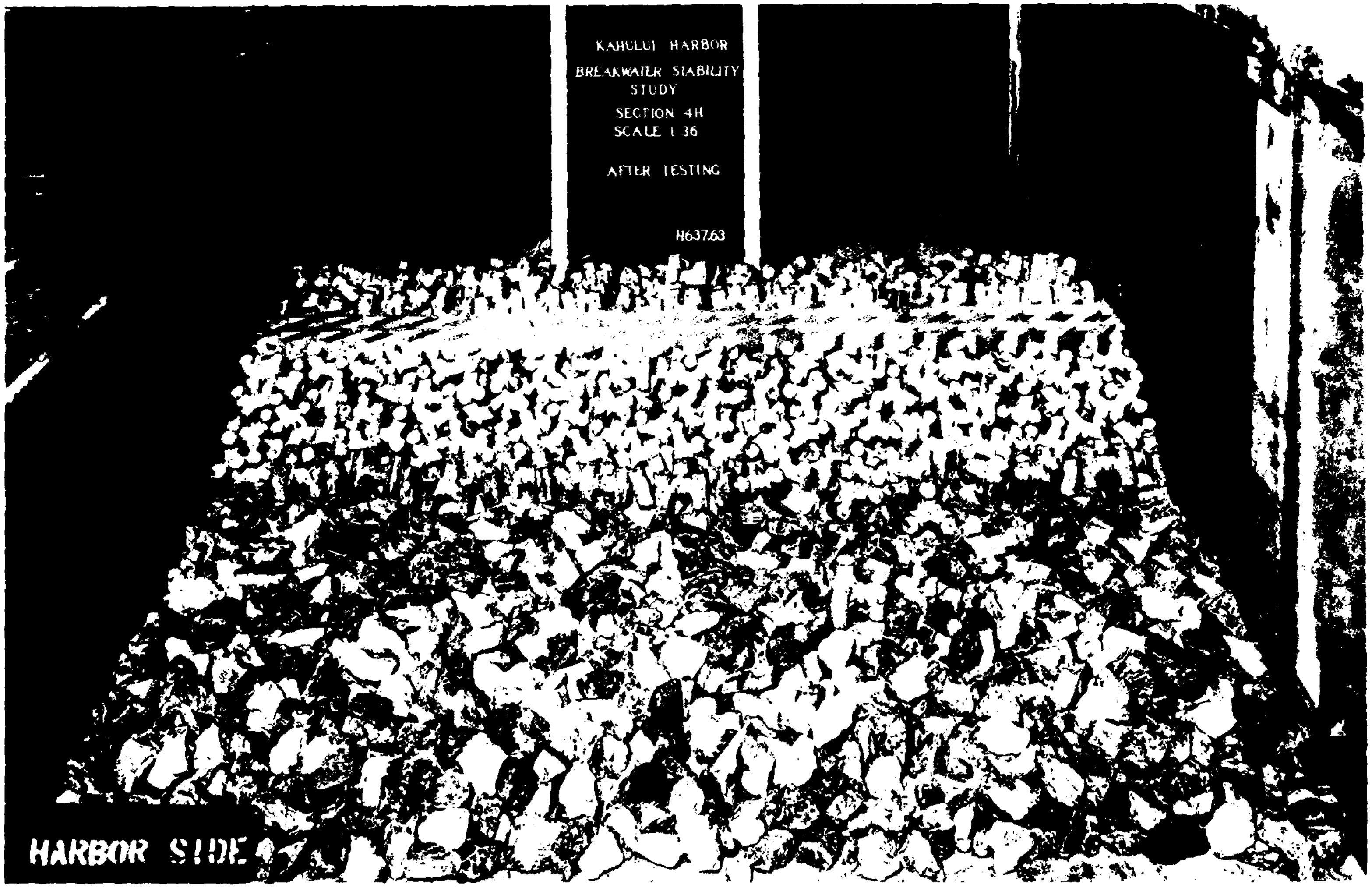


Photo 47. Harbor-side view of Plan 1H after testing

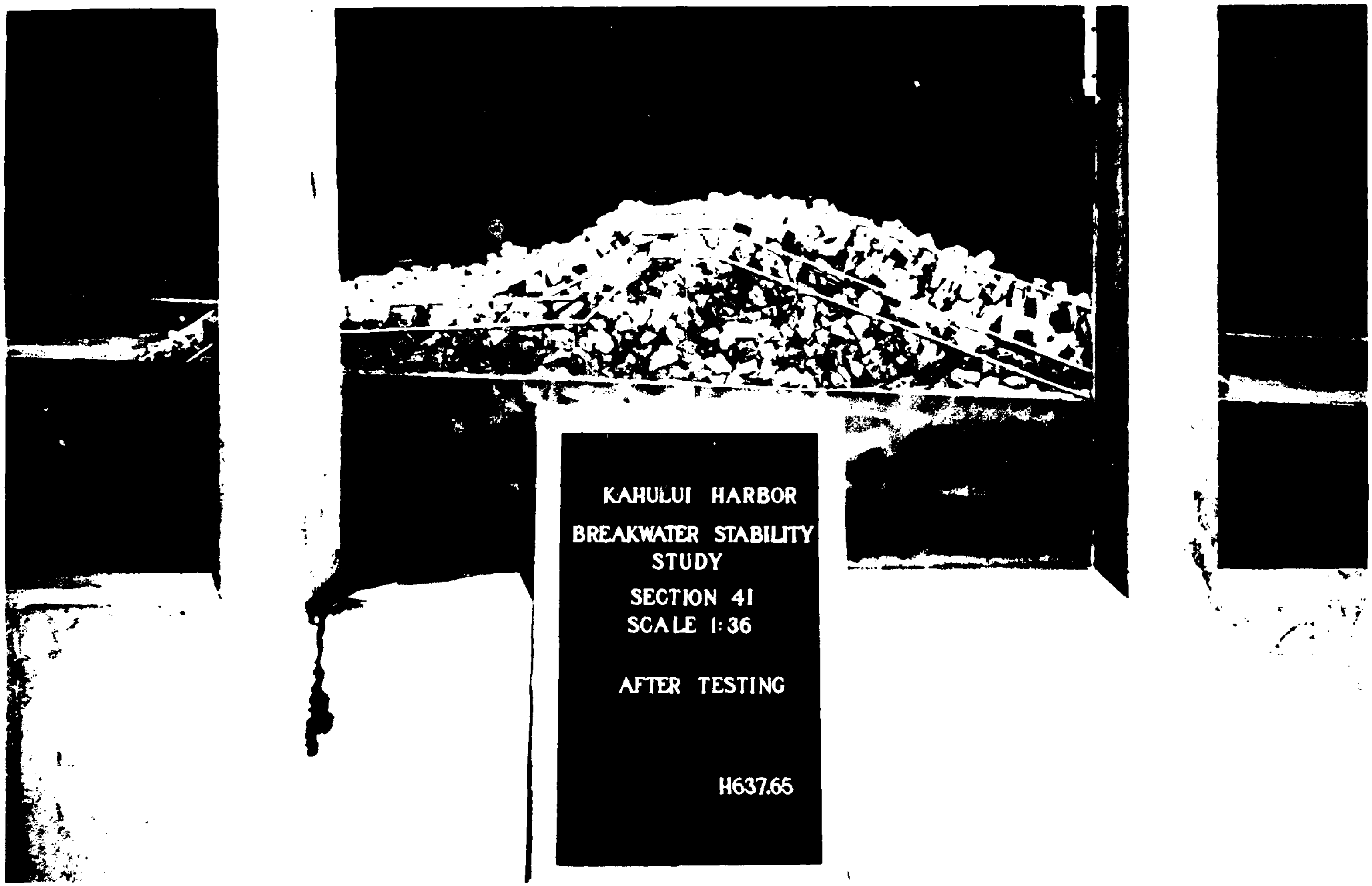


Photo 43 Side view of Plan II after testing

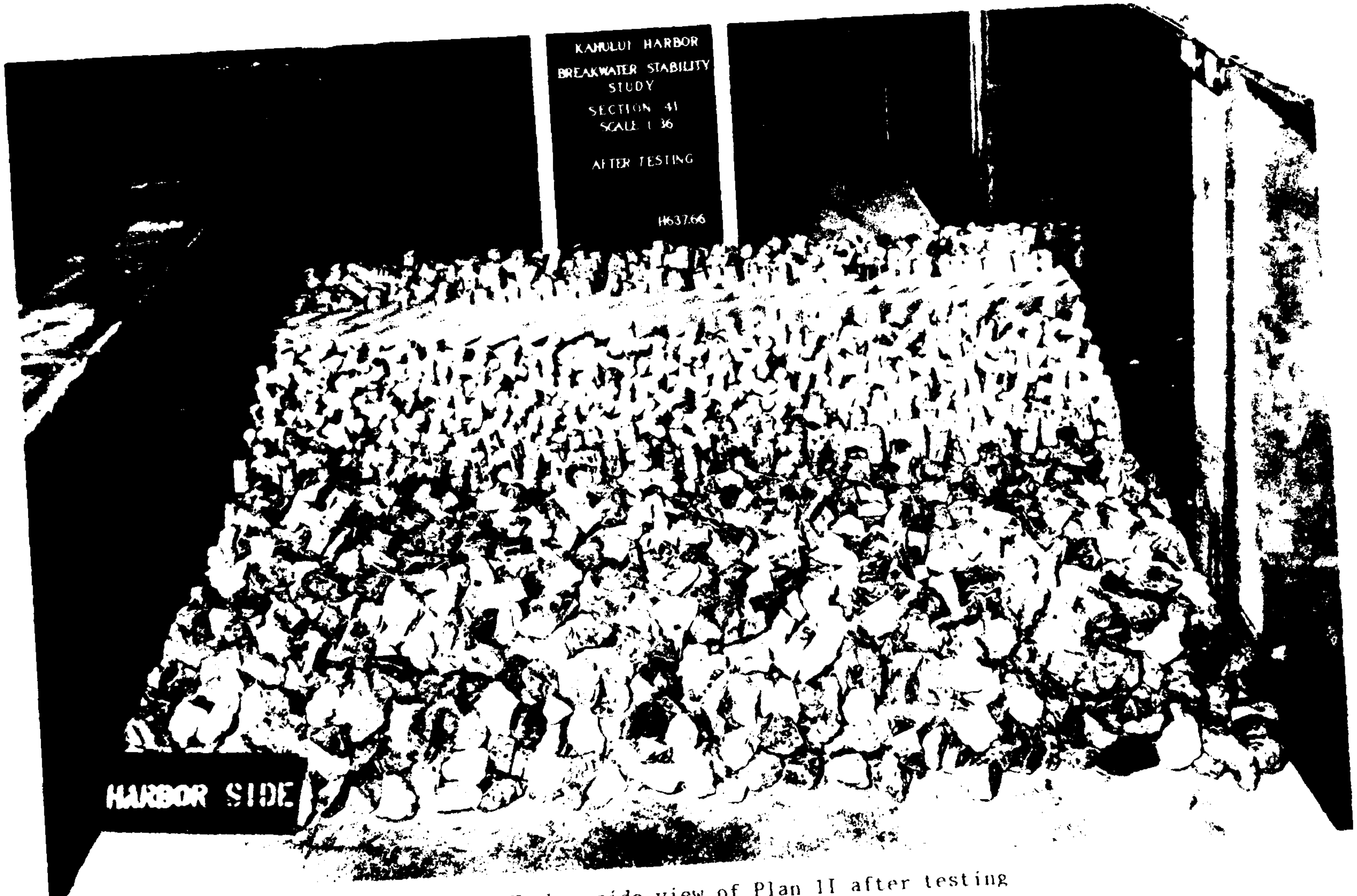


Photo 49. Harbor-side view of Plan II after testing

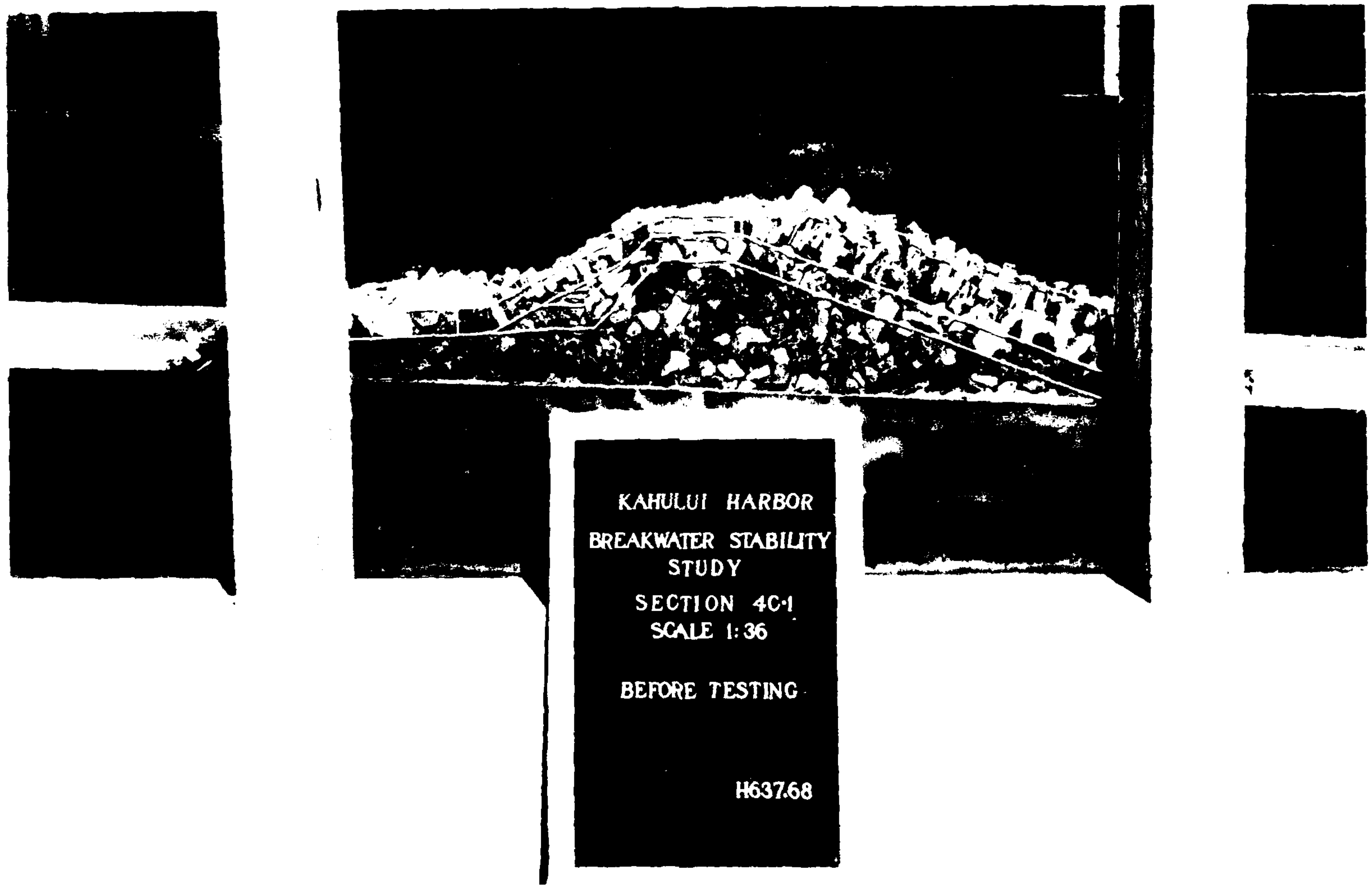


Photo 50. Side view of Plan 1F-1 before testing, 2nd test

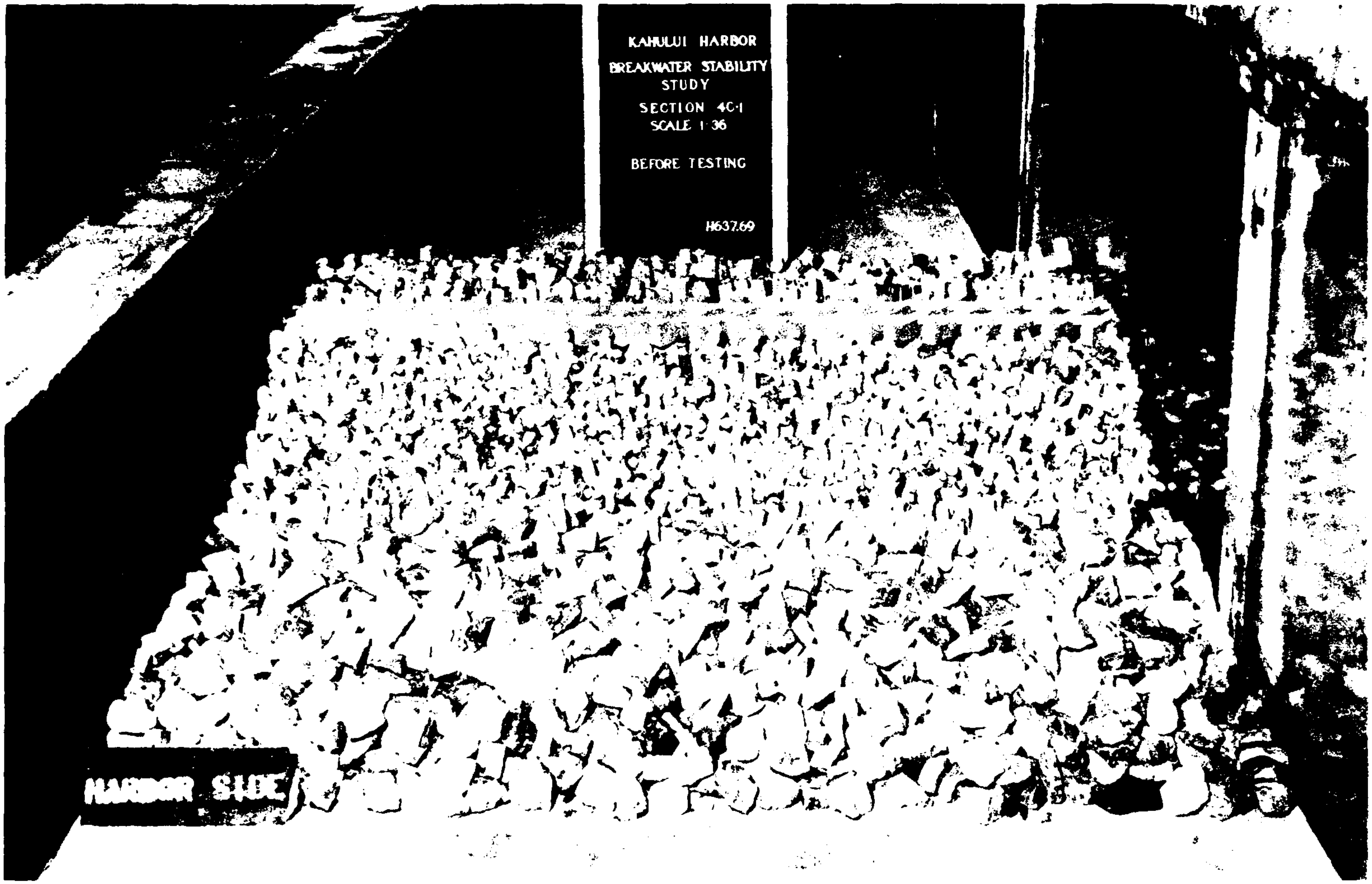


Photo 51. Harbor-side view of Plan 1F-1 before testing, 2nd test

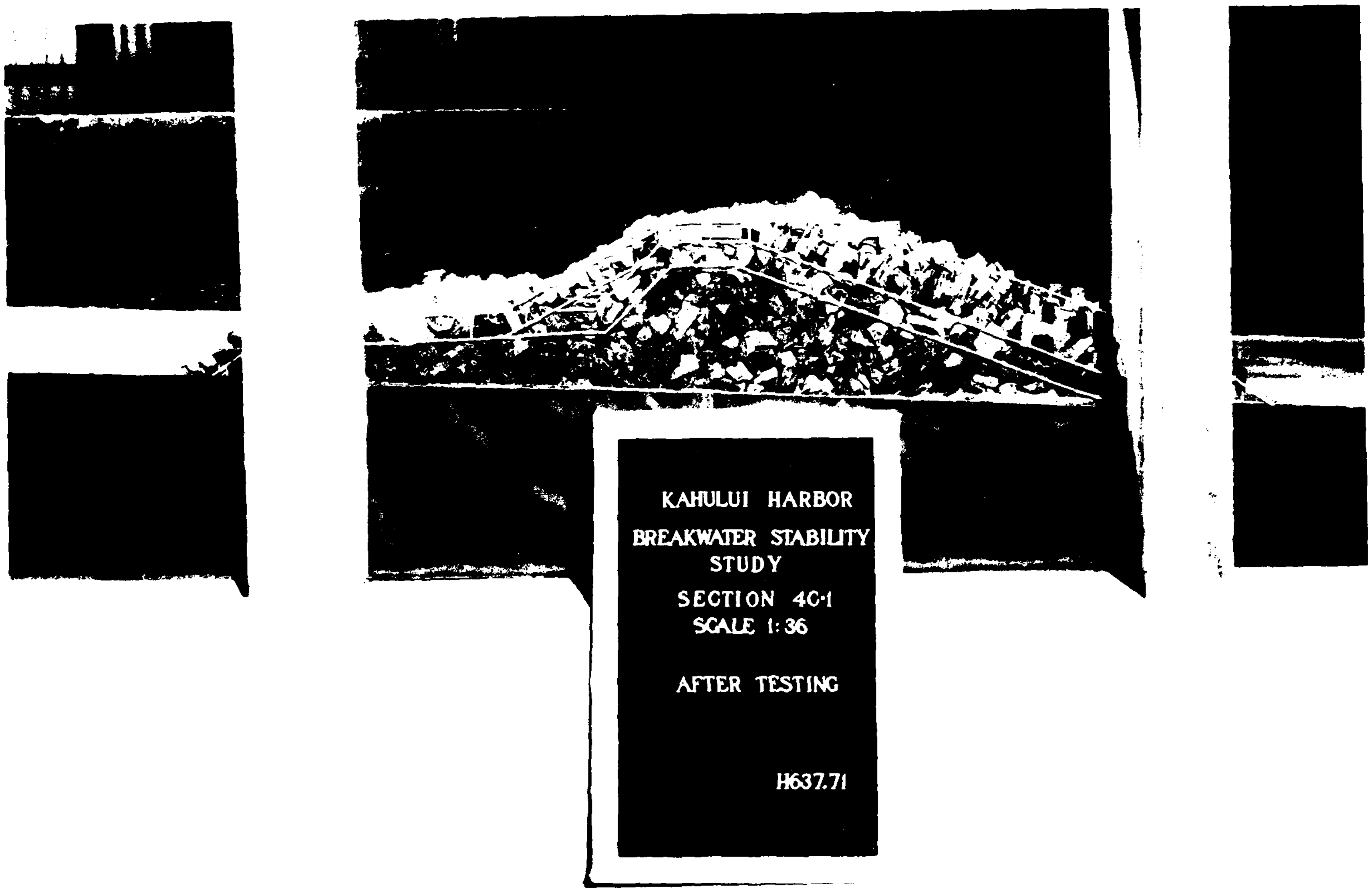


Photo 52. Side view of Plan 1F-1 after testing, 2nd test

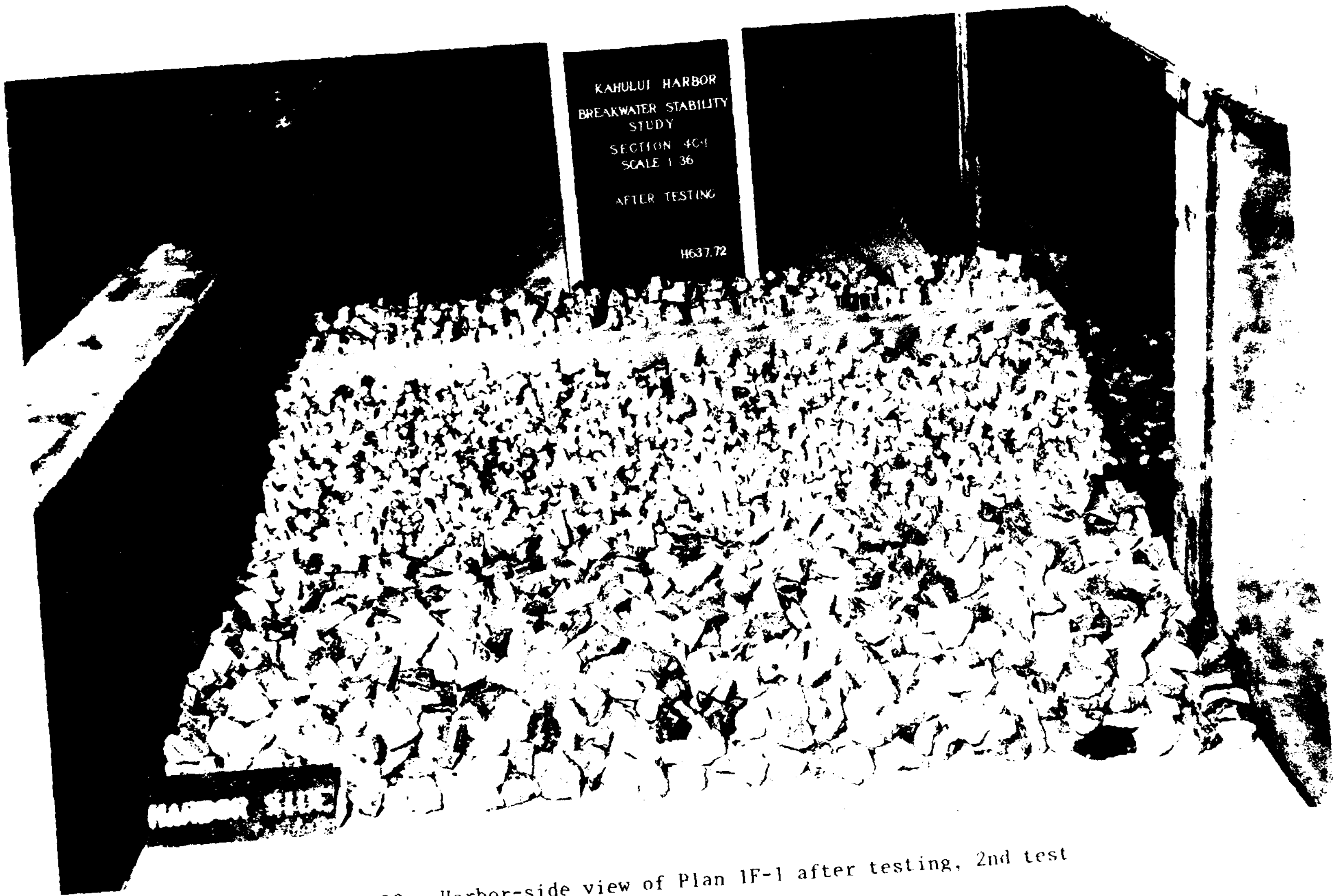


Photo 53. Harbor-side view of Plan 1F-1 after testing, 2nd test

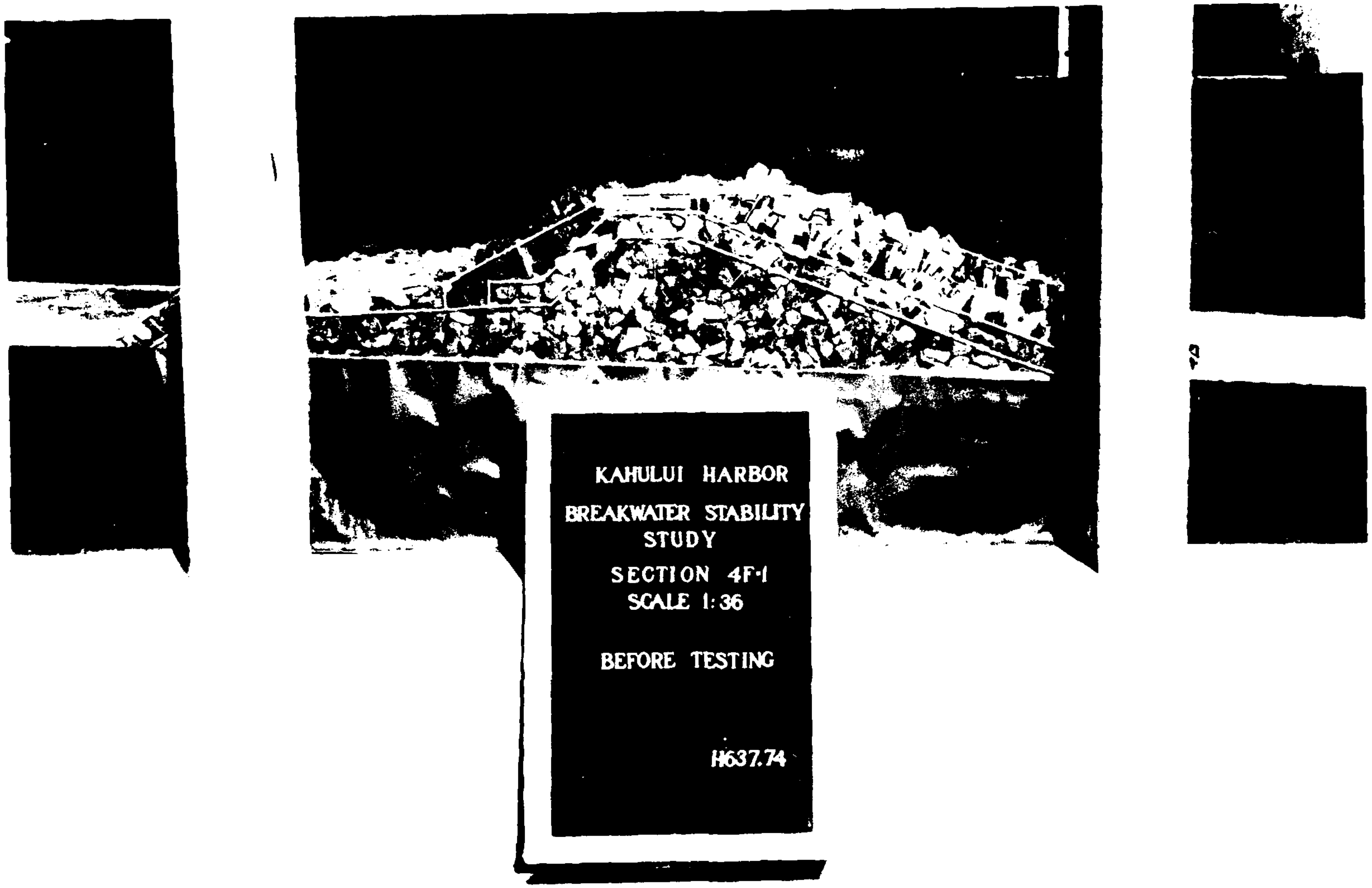


Photo 54. Side view of Plan 1D-2 before testing, 2nd test

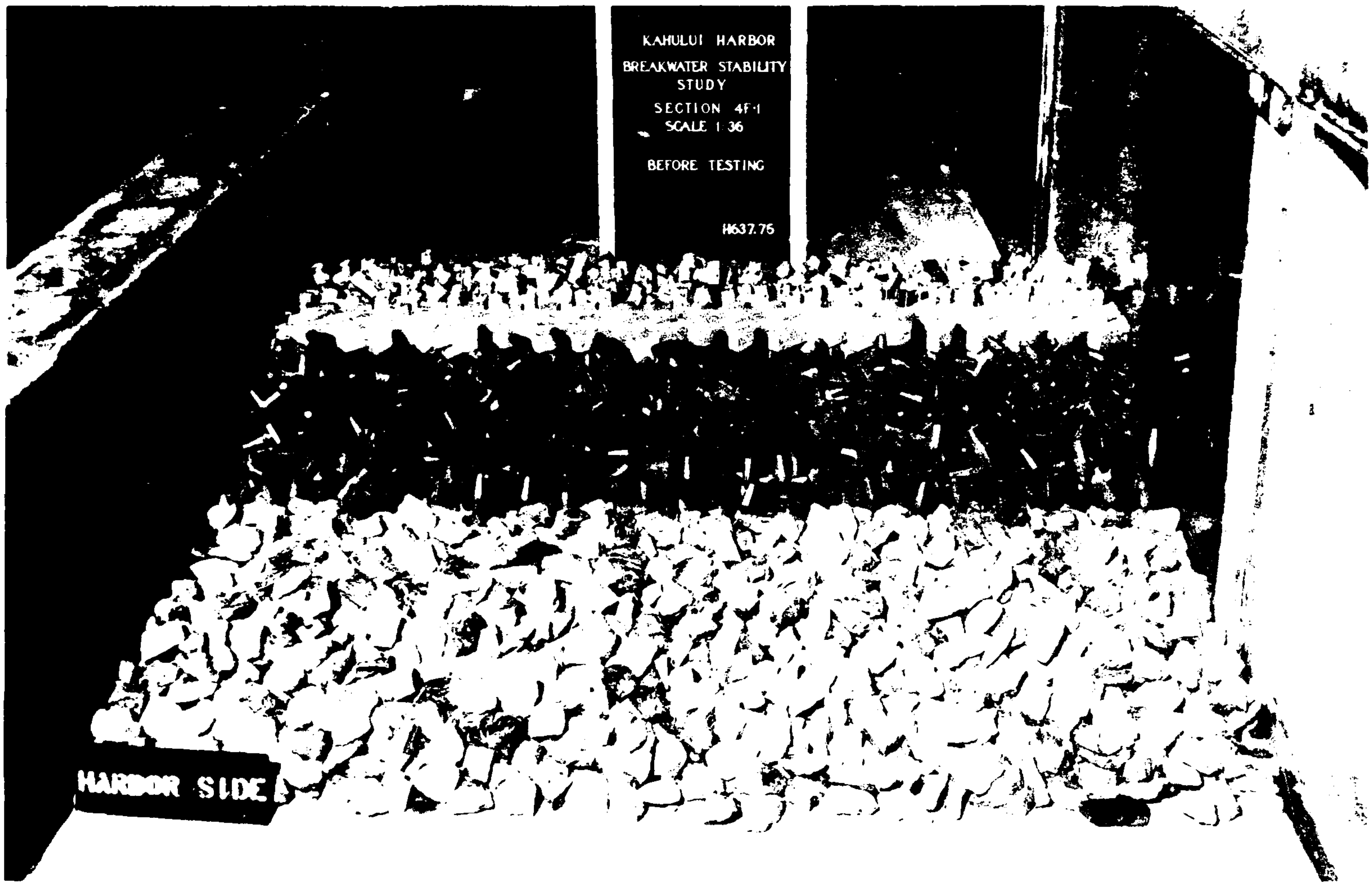


Photo 55. Harbor-side view of Plan 1D-2 before testing, 2nd test

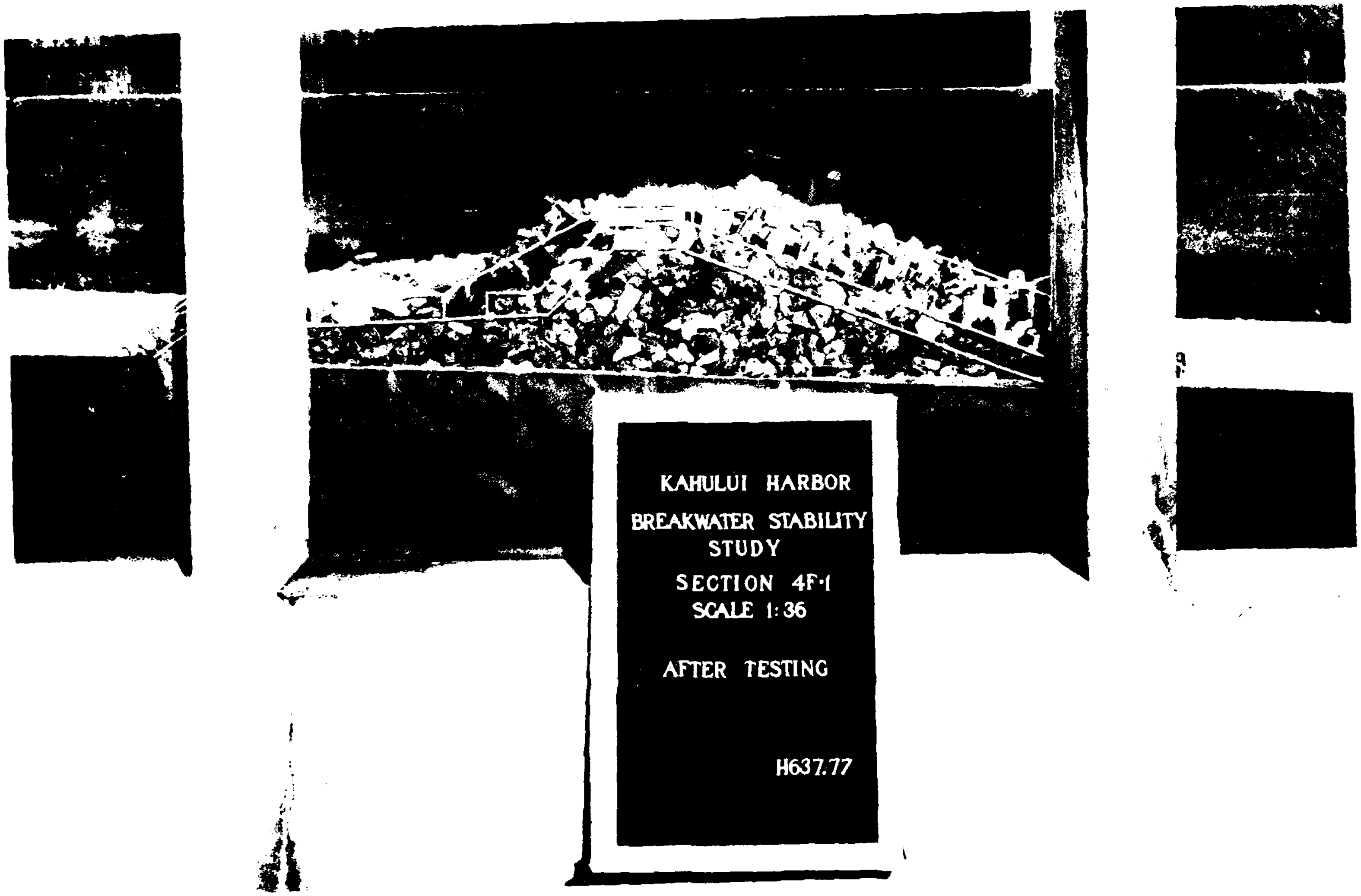


Photo 56. Side view of Plan 1D-2 after testing, 2nd test

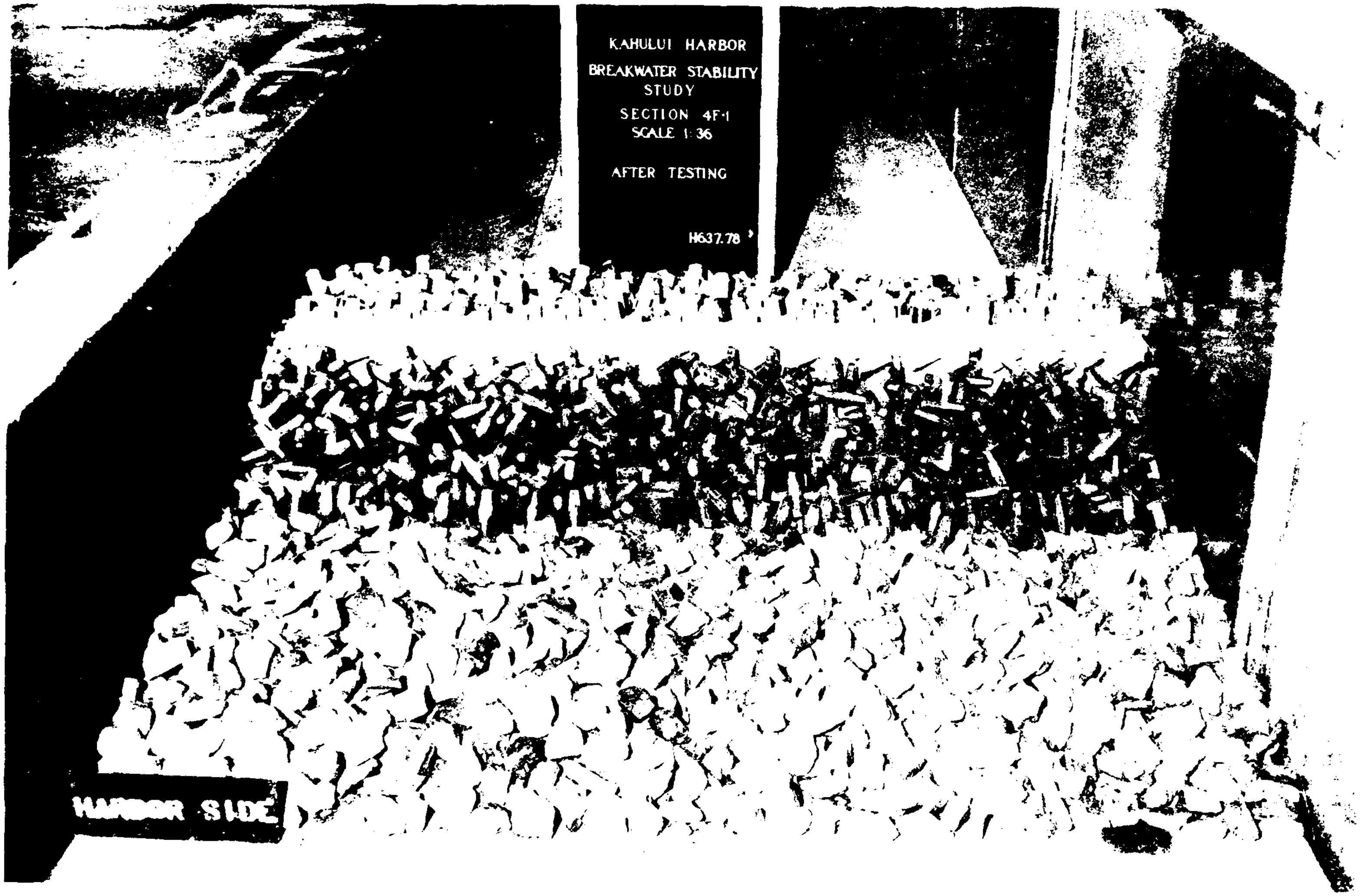


Photo 57. Harbor-side view of Plan 1D-2 after testing, 2nd test

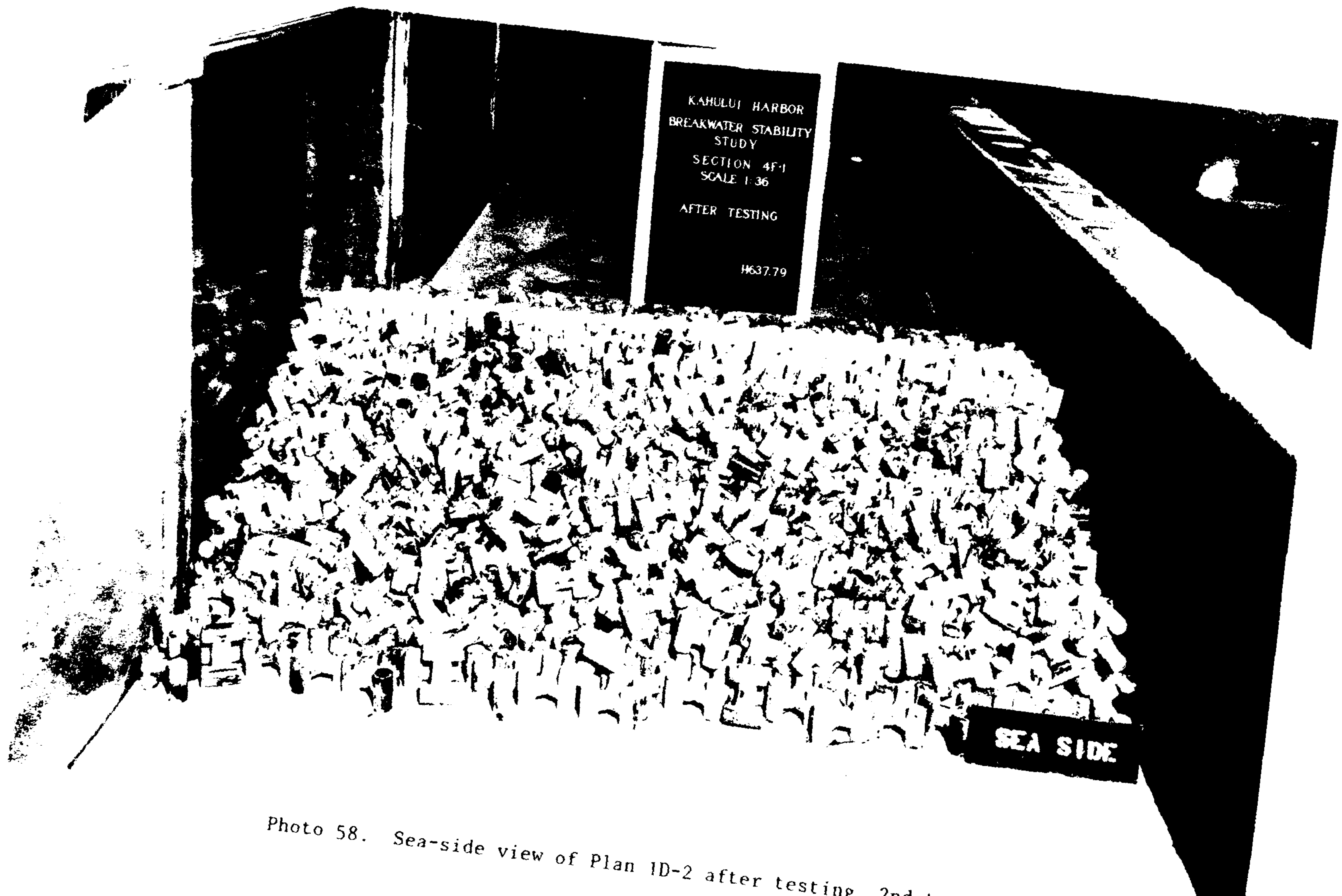


Photo 58. Sea-side view of Plan 1D-2 after testing, 2nd test

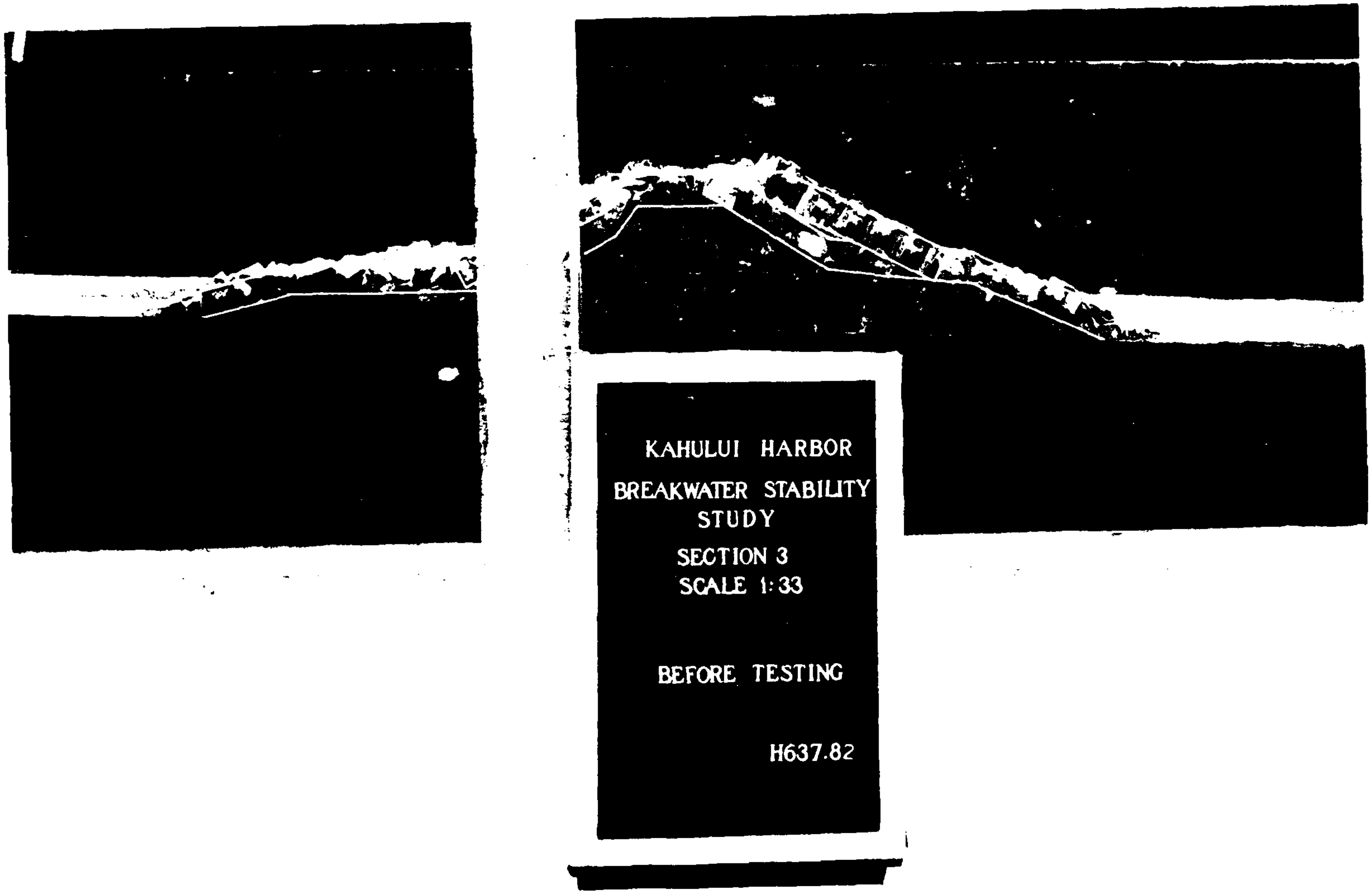


Photo 59. Side view of Plan 2 before testing

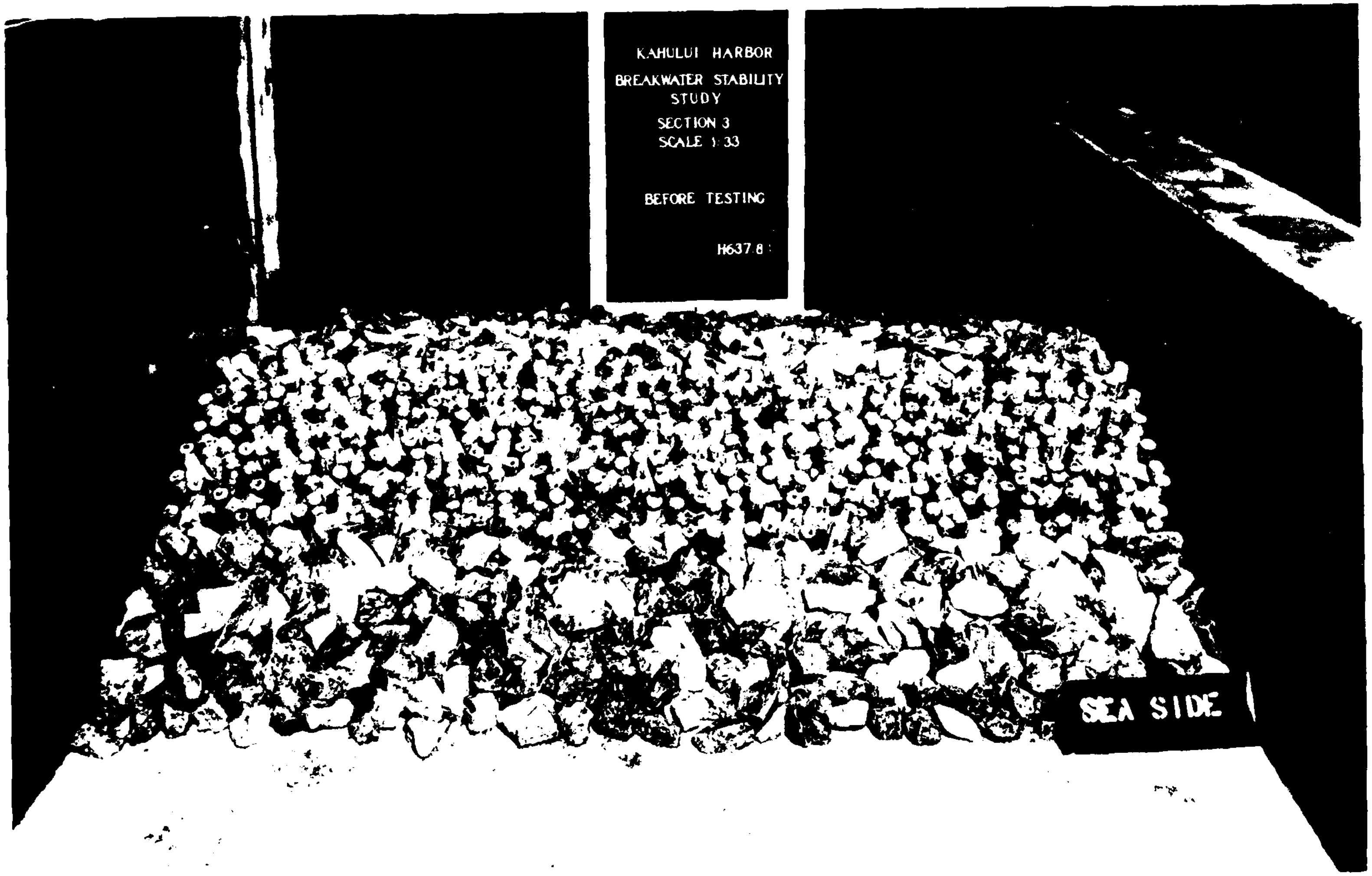


Photo 60. Sea-side view of Plan 2 before testing

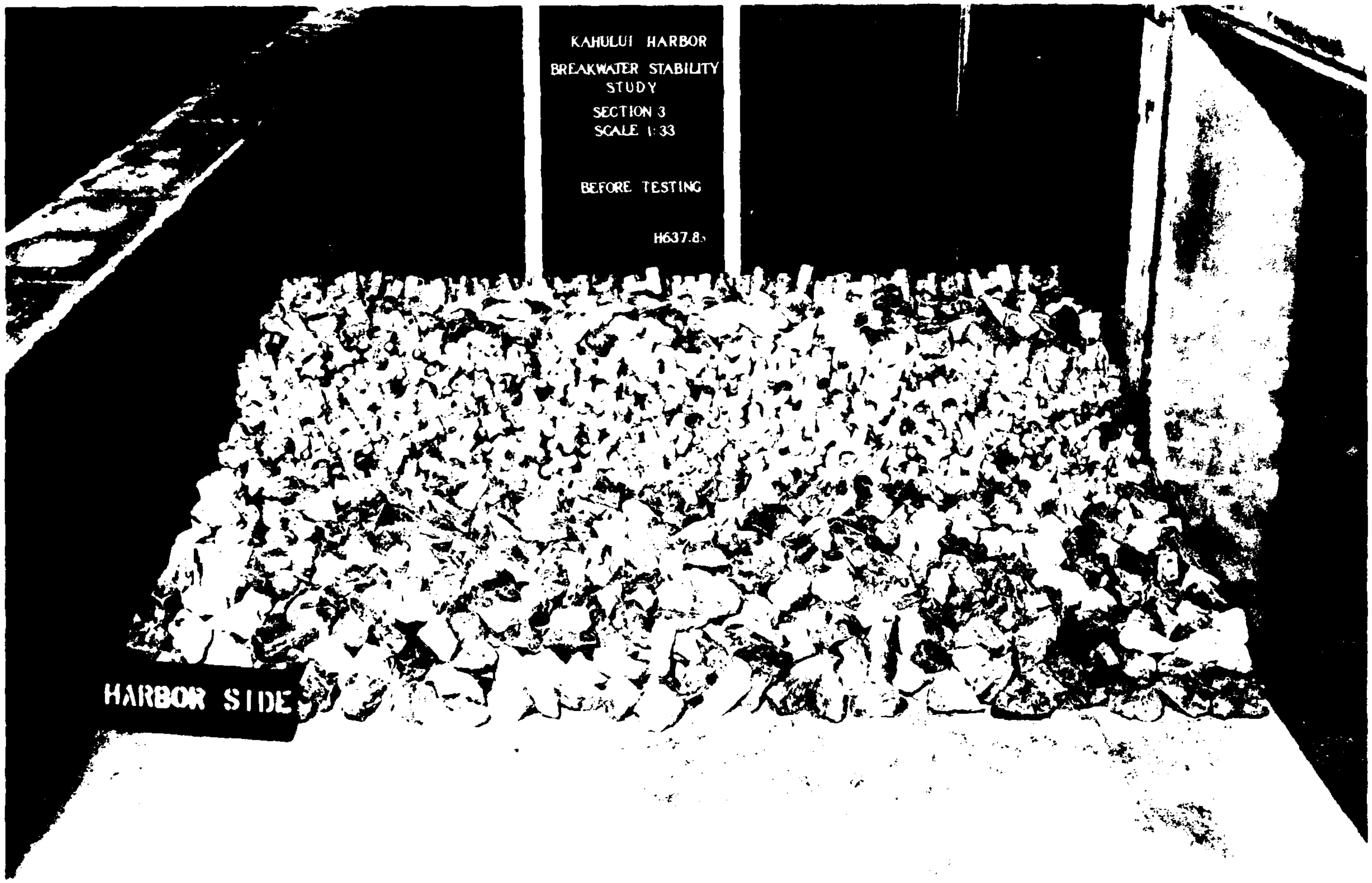


Photo 61. Harbor-side view of Plan 2 before testing

AD-A120 181 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/2
KAHULUI BREAKWATER STABILITY STUDY, KAHULUI, MAUI, HAWAII. HYDR--ETC(U)
JUL 82 D G MARKLE
UNCLASSIFIED WES/TR/HL-82-14 NL

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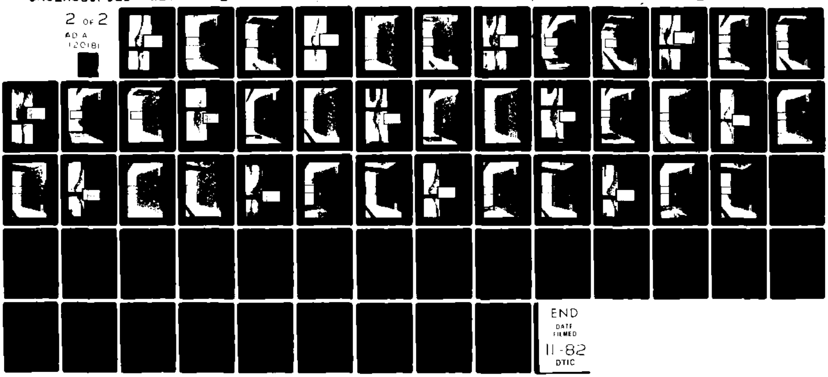


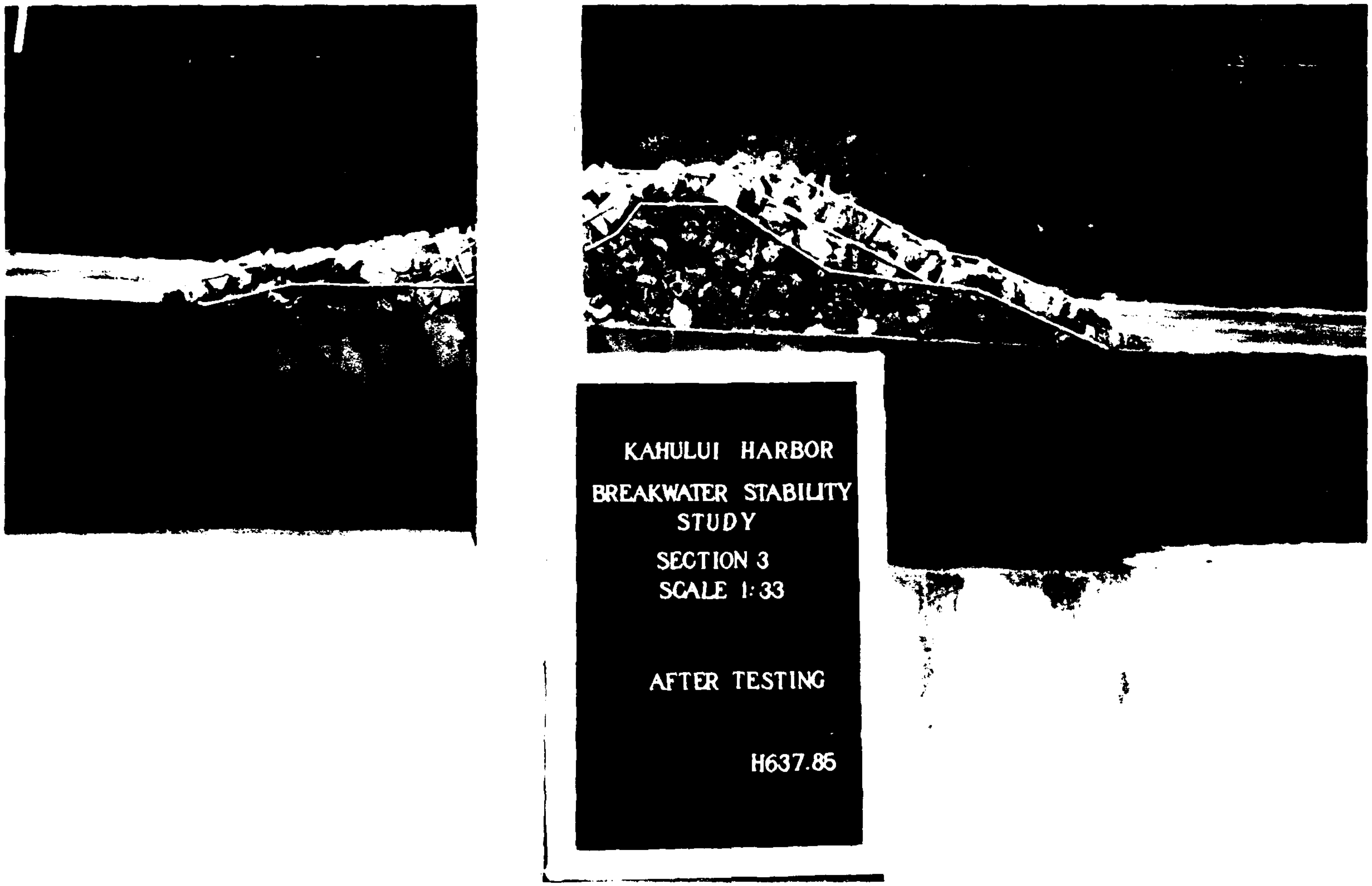


Photo 62. Side view of Plan 2 after testing, 1st test

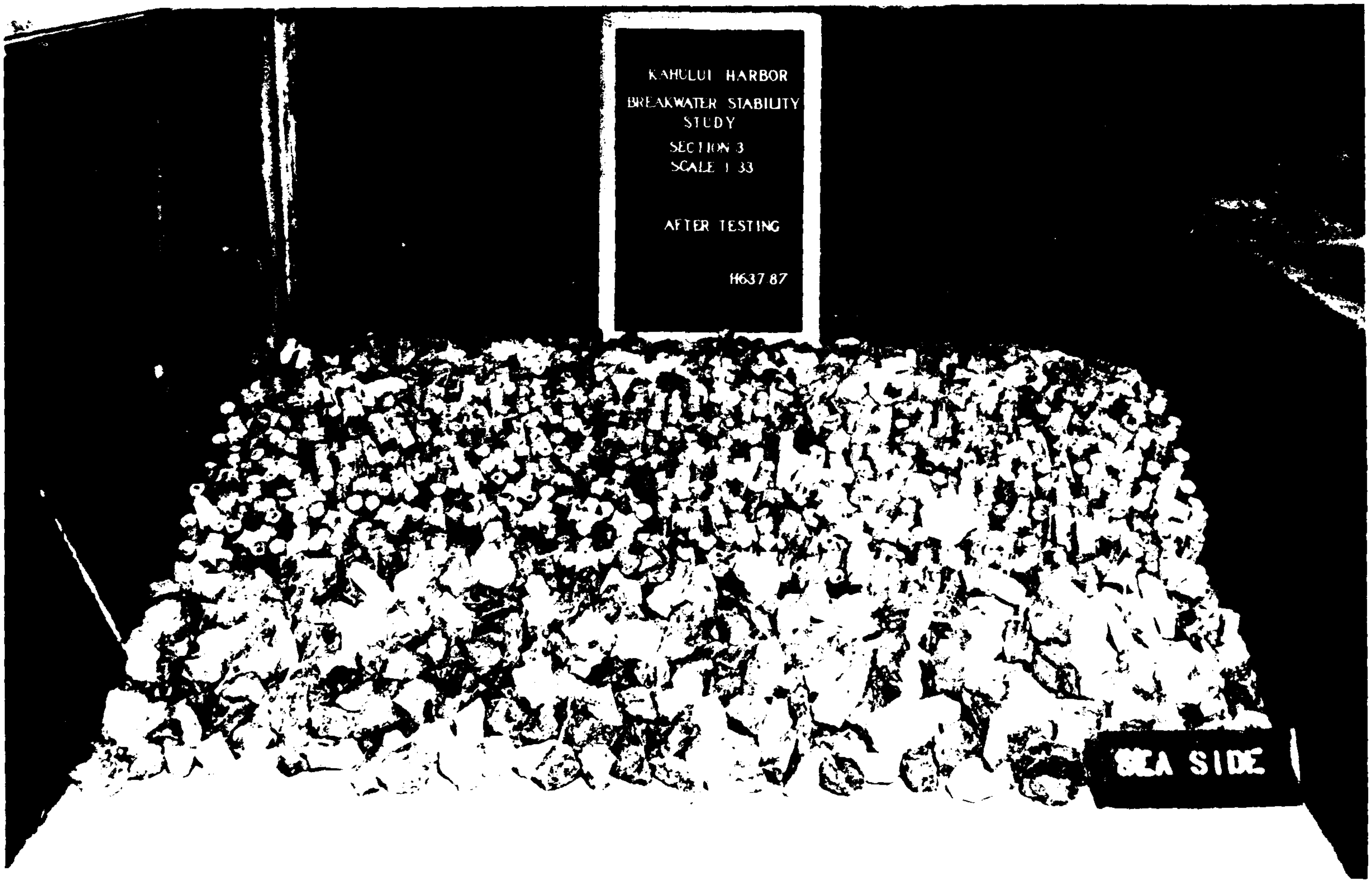


Photo 63. Sea-side view of Plan 2 after testing, 1st test

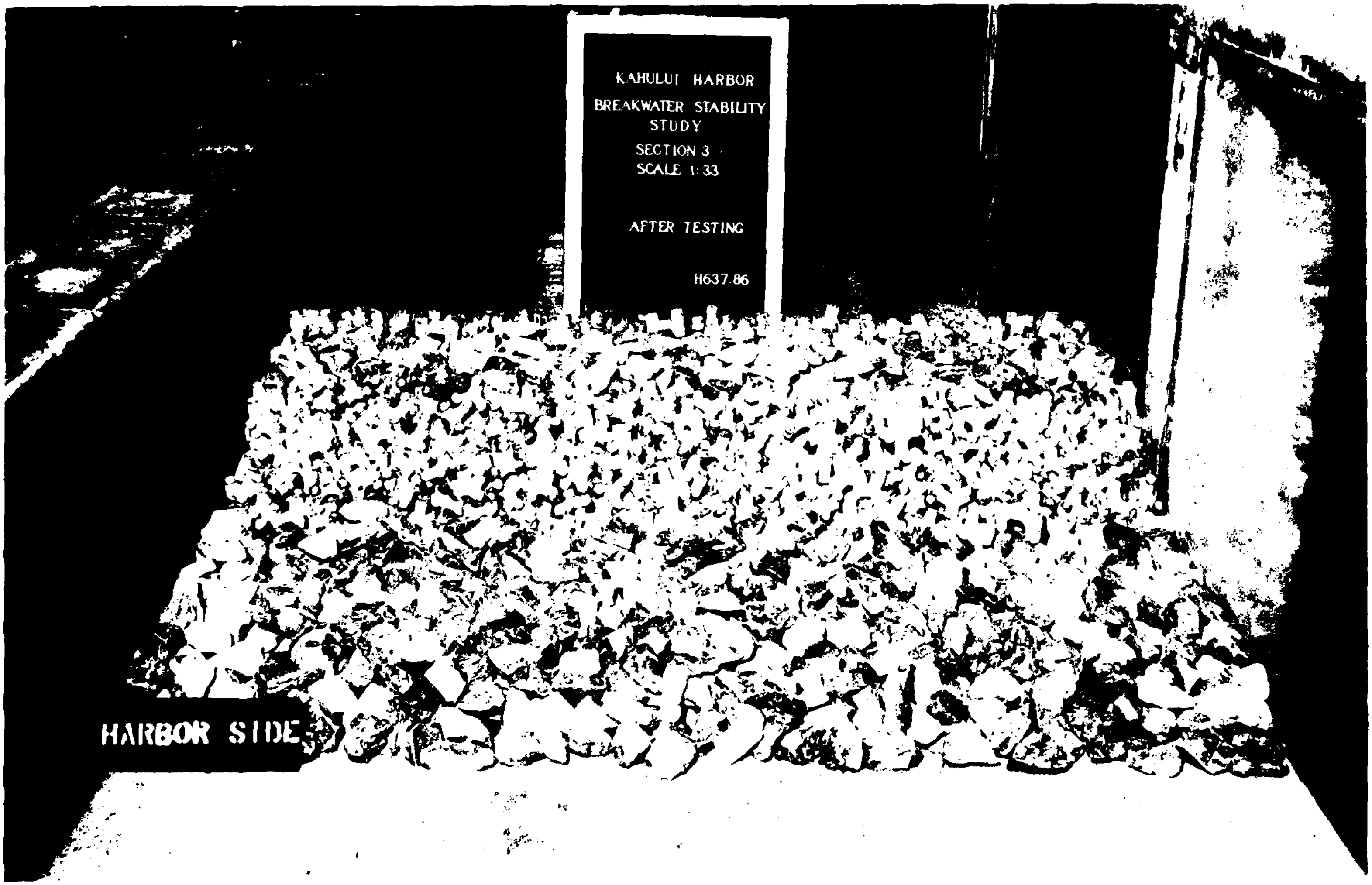


Photo 64. Harbor-side view of Plan 2 after testing, 1st test

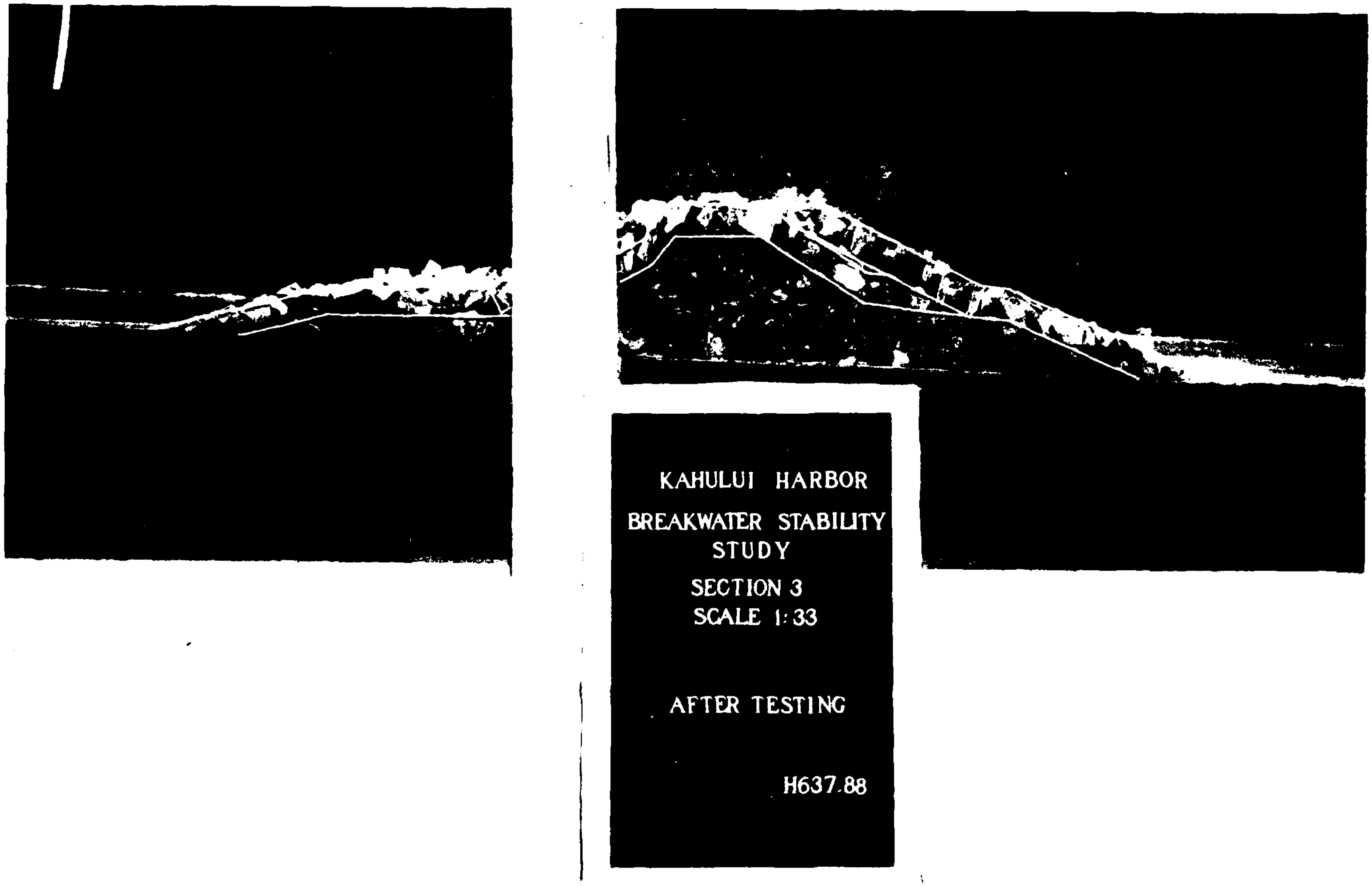


Photo 65. Side view of Plan 2 after testing, 2nd test

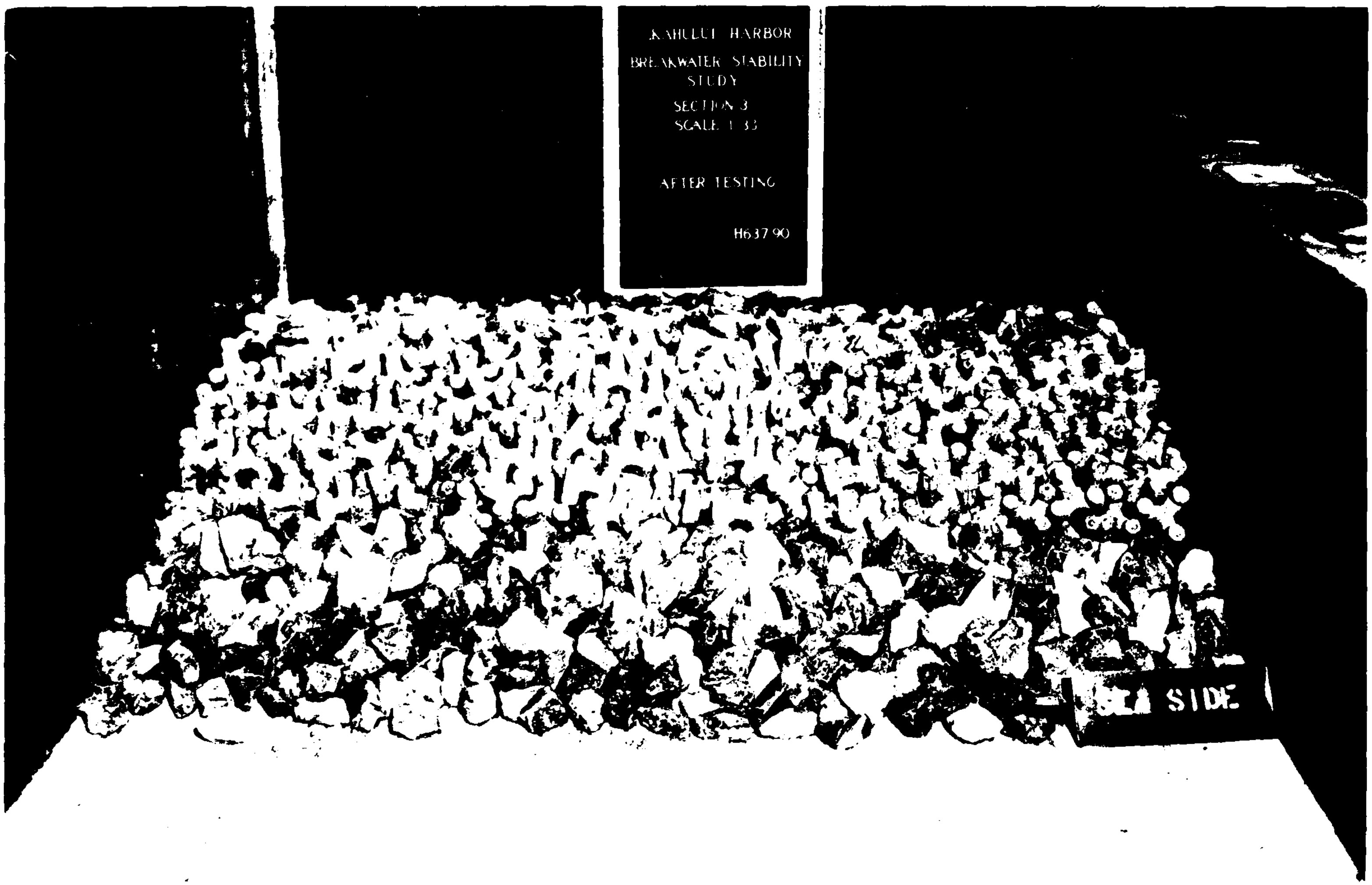


Photo 66. Sea-side view of Plan 2 after testing, 2nd test

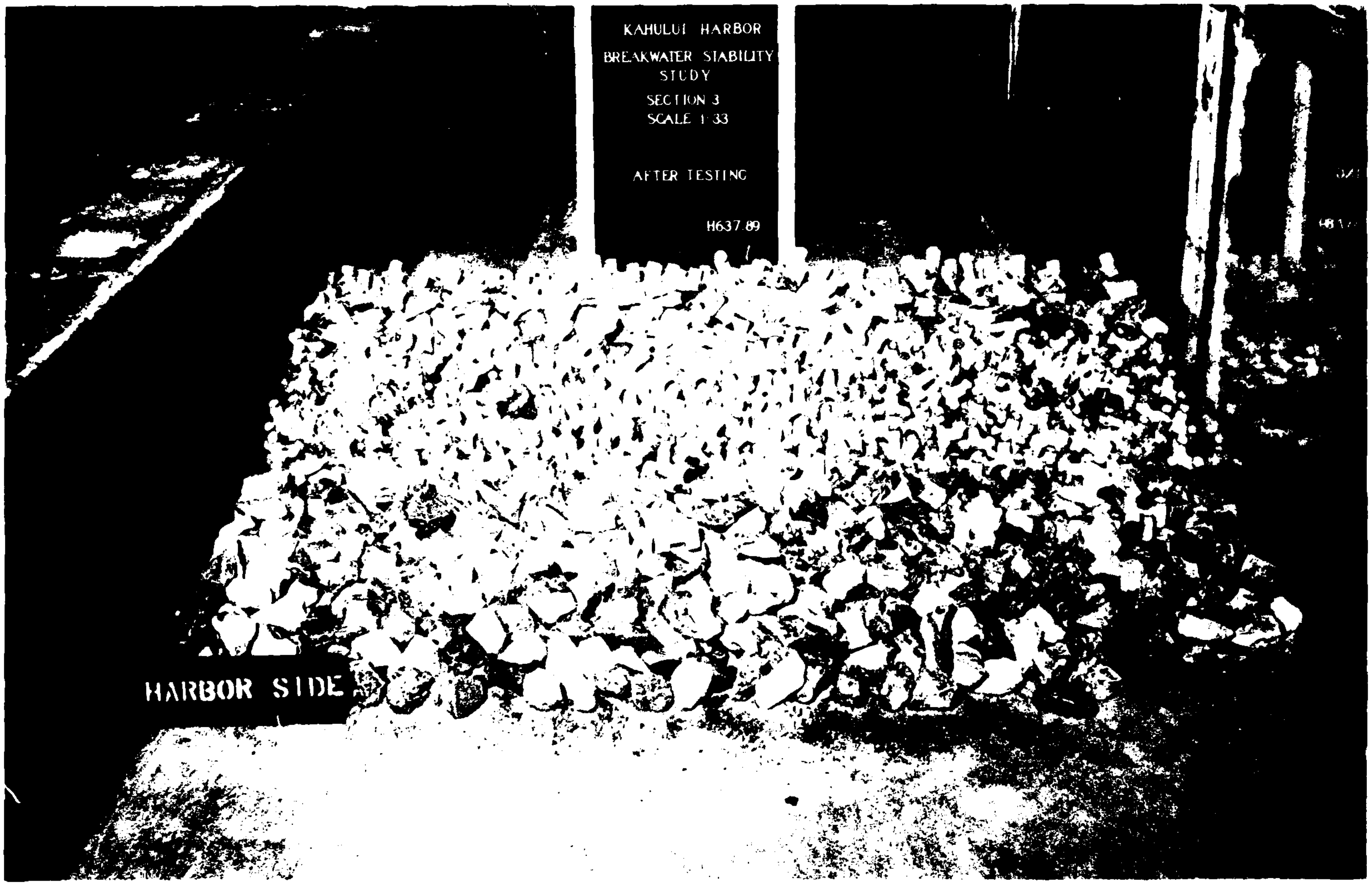


Photo 67. Harbor-side view of Plan 2 after testing, 2nd test

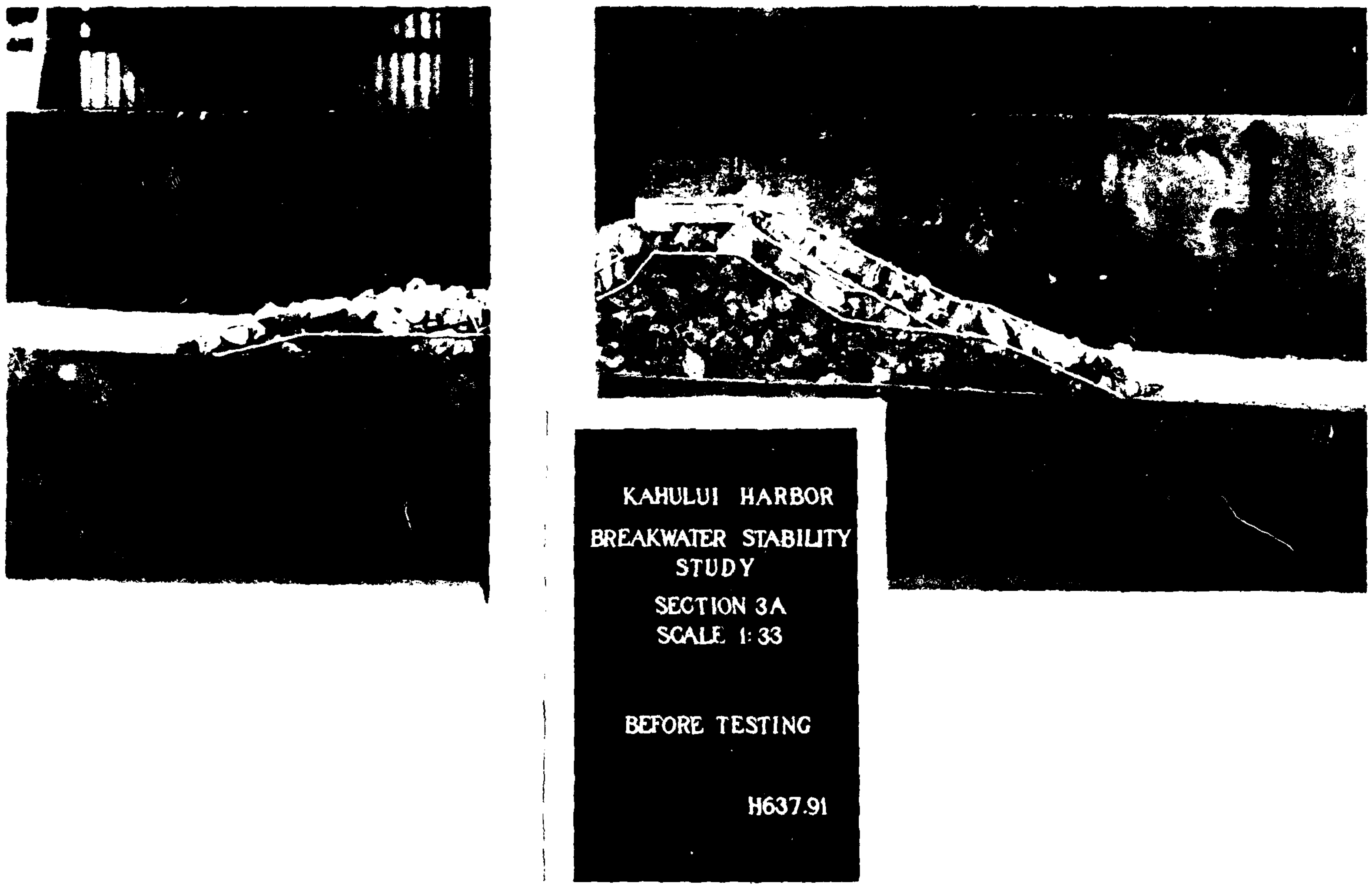


Photo 68. Side view of Plan 2A before testing

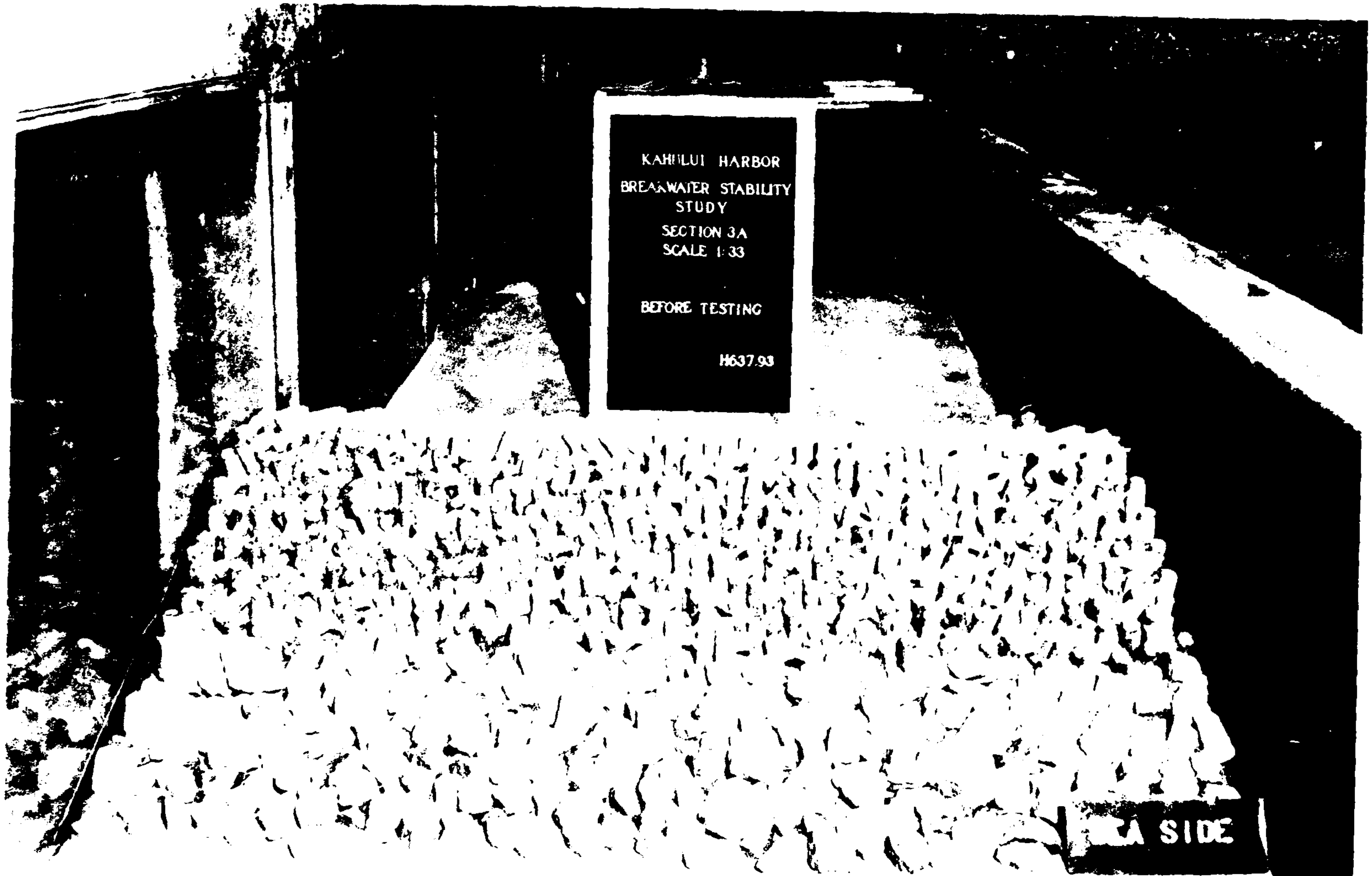


Photo 69. Sea-side view of Plan 2A before testing

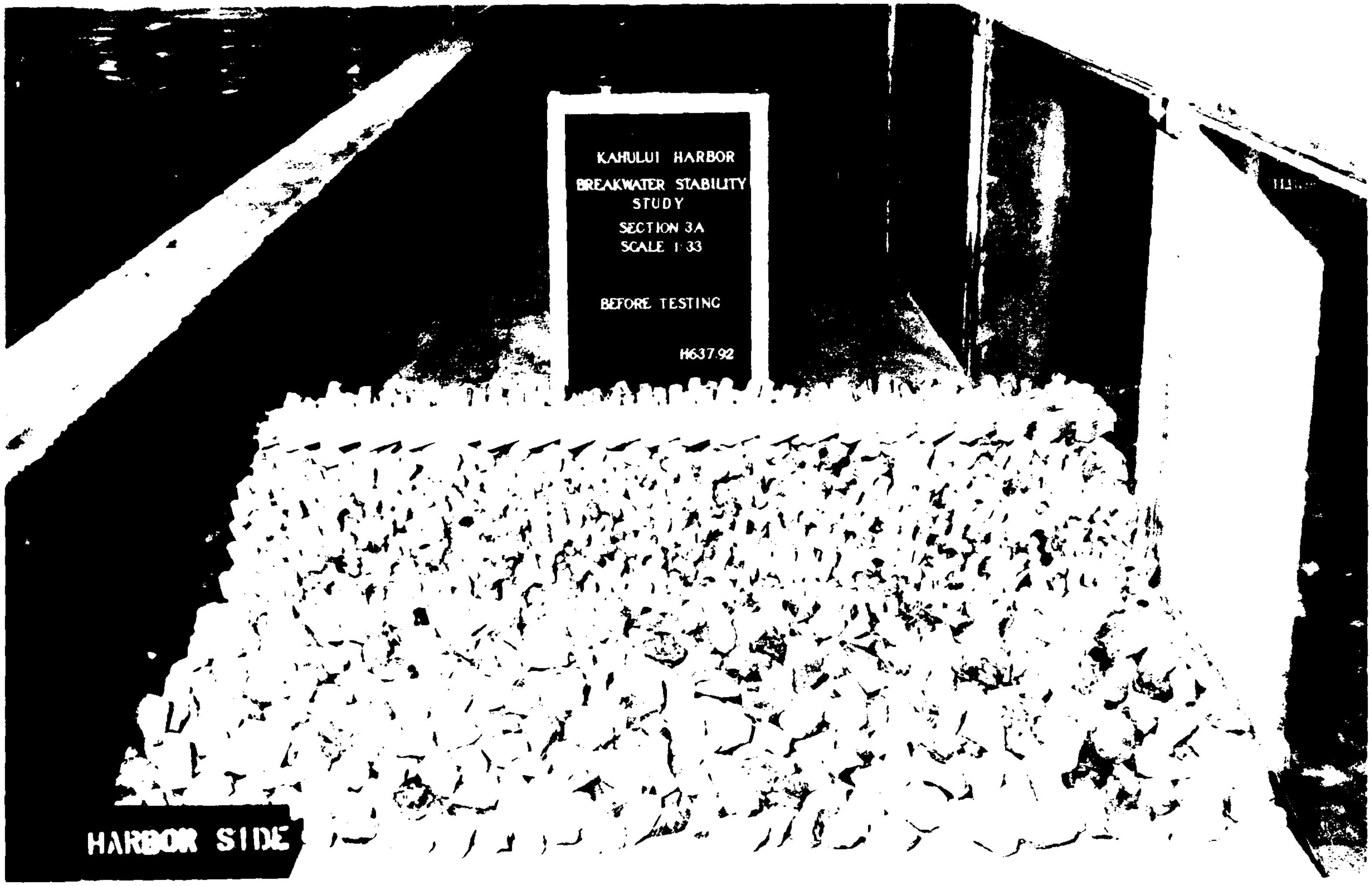


Photo 70. Harbor-side view of Plan 2A before testing

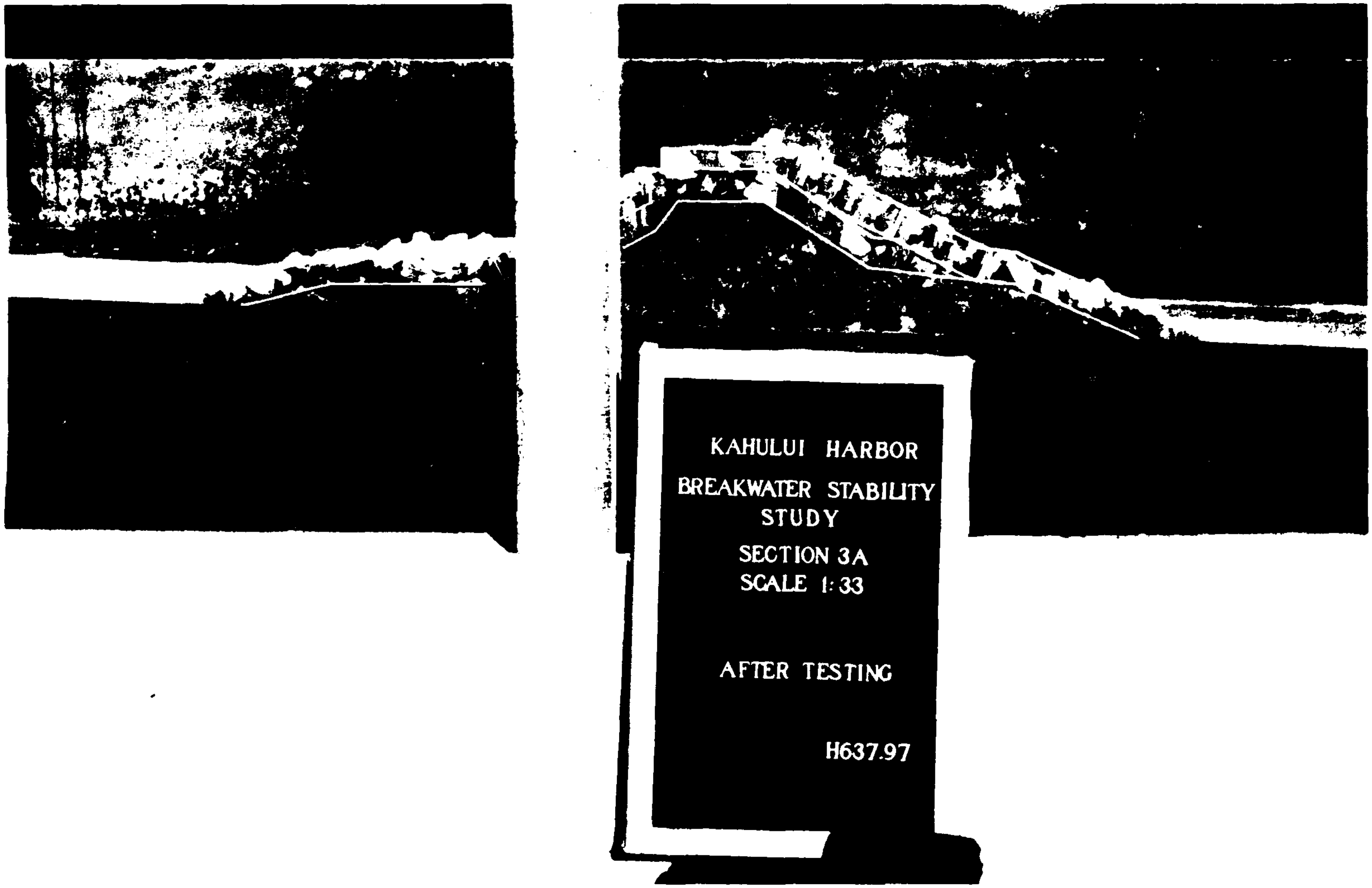


Photo 71. Side view of Plan 2A after testing, 1st test

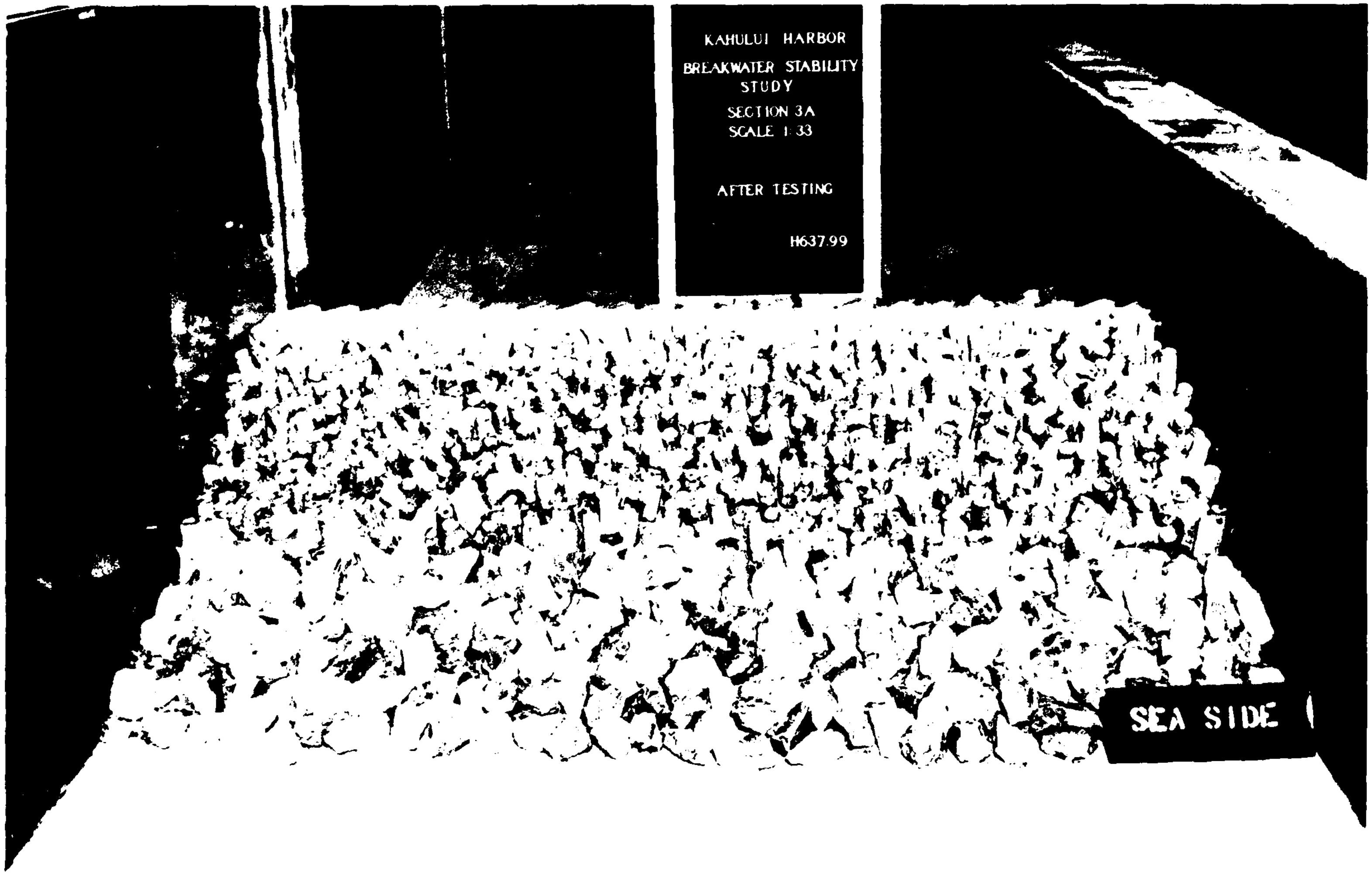


Photo 72. Sea-side view of Plan ?A after te?ting, 1st test

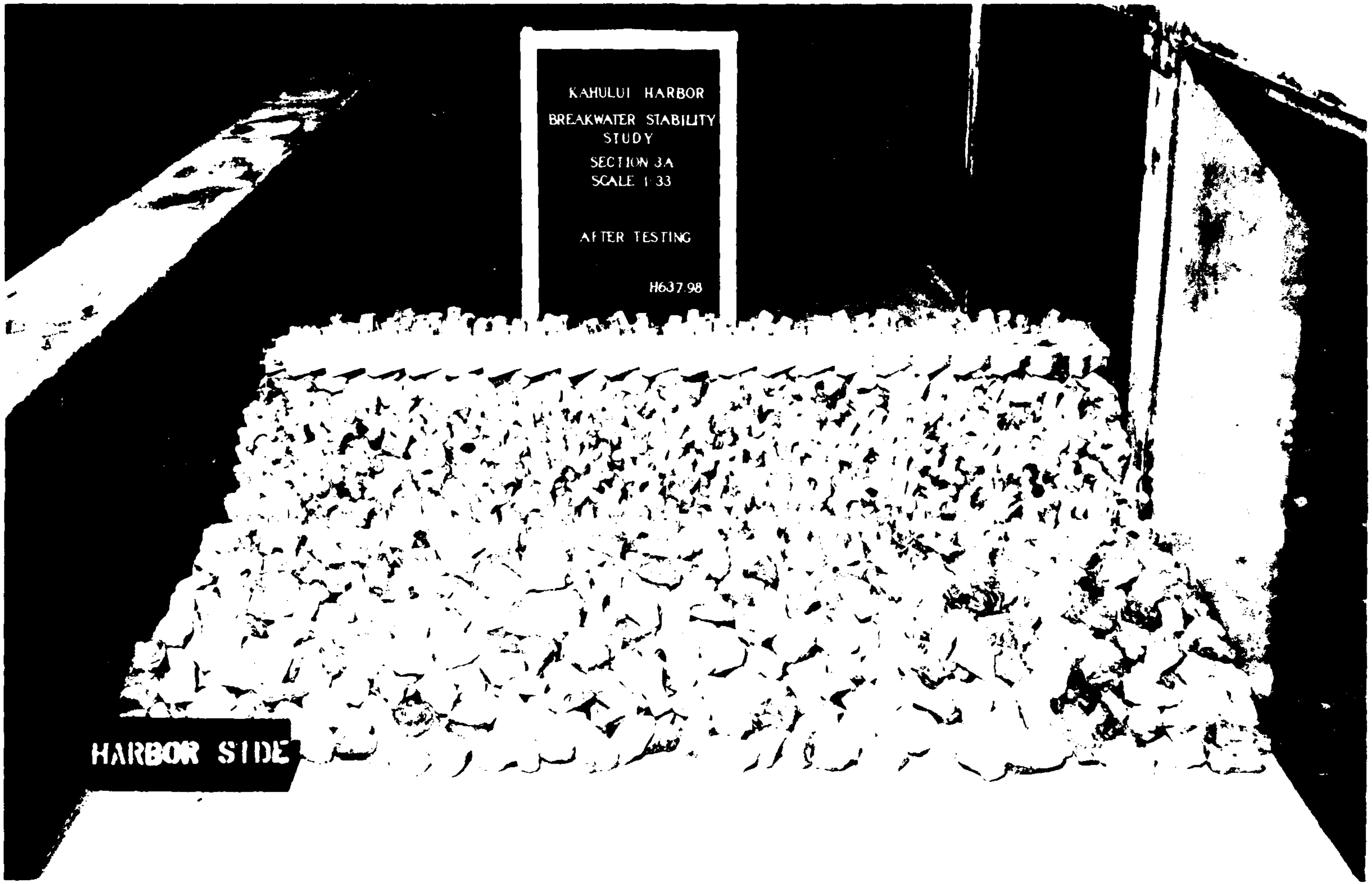


Photo 73. Harbor-side view of Plan 2A after testing, 1st test

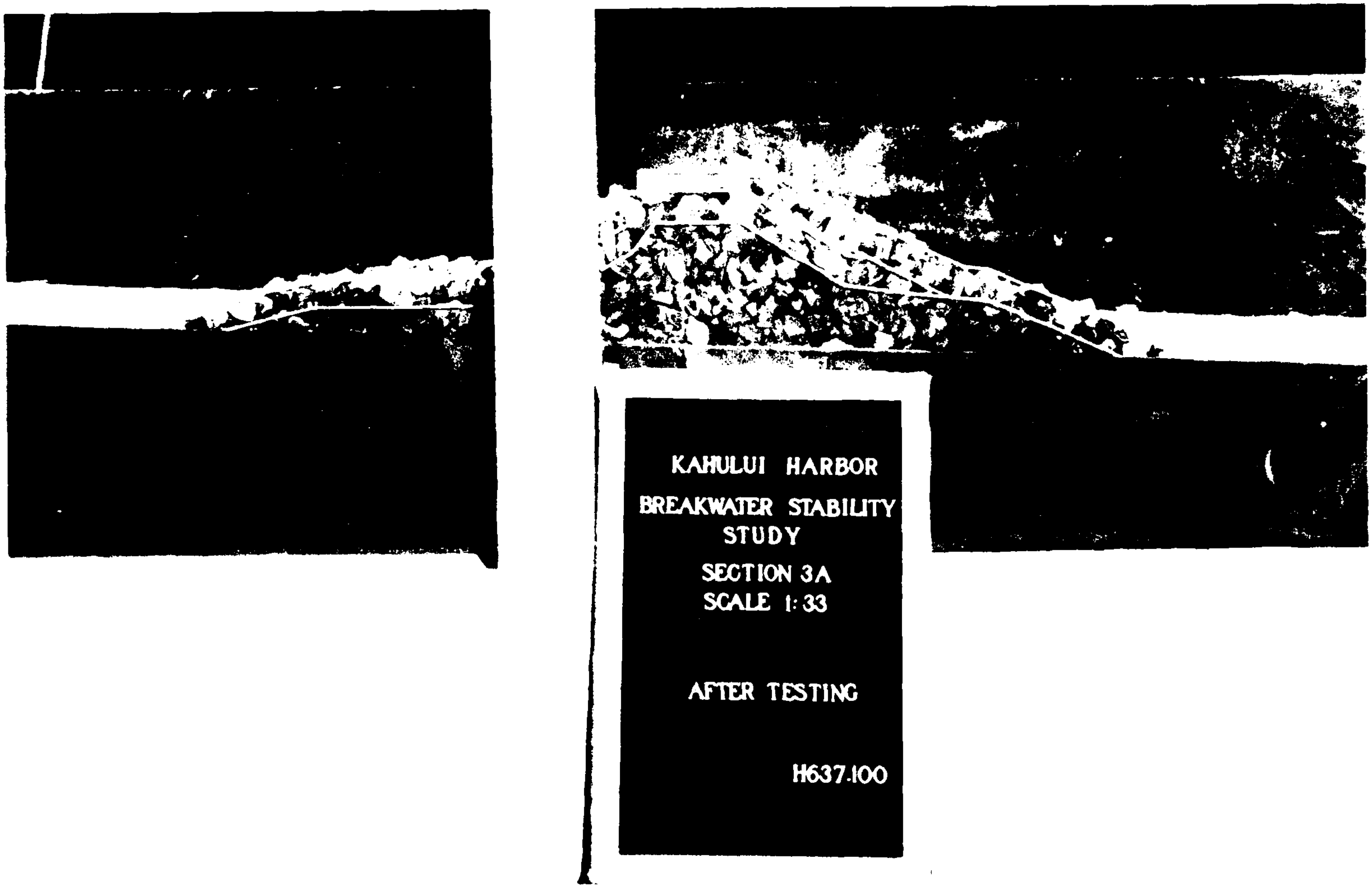


Photo 74. Side view of Plan 2A after testing, 2nd test

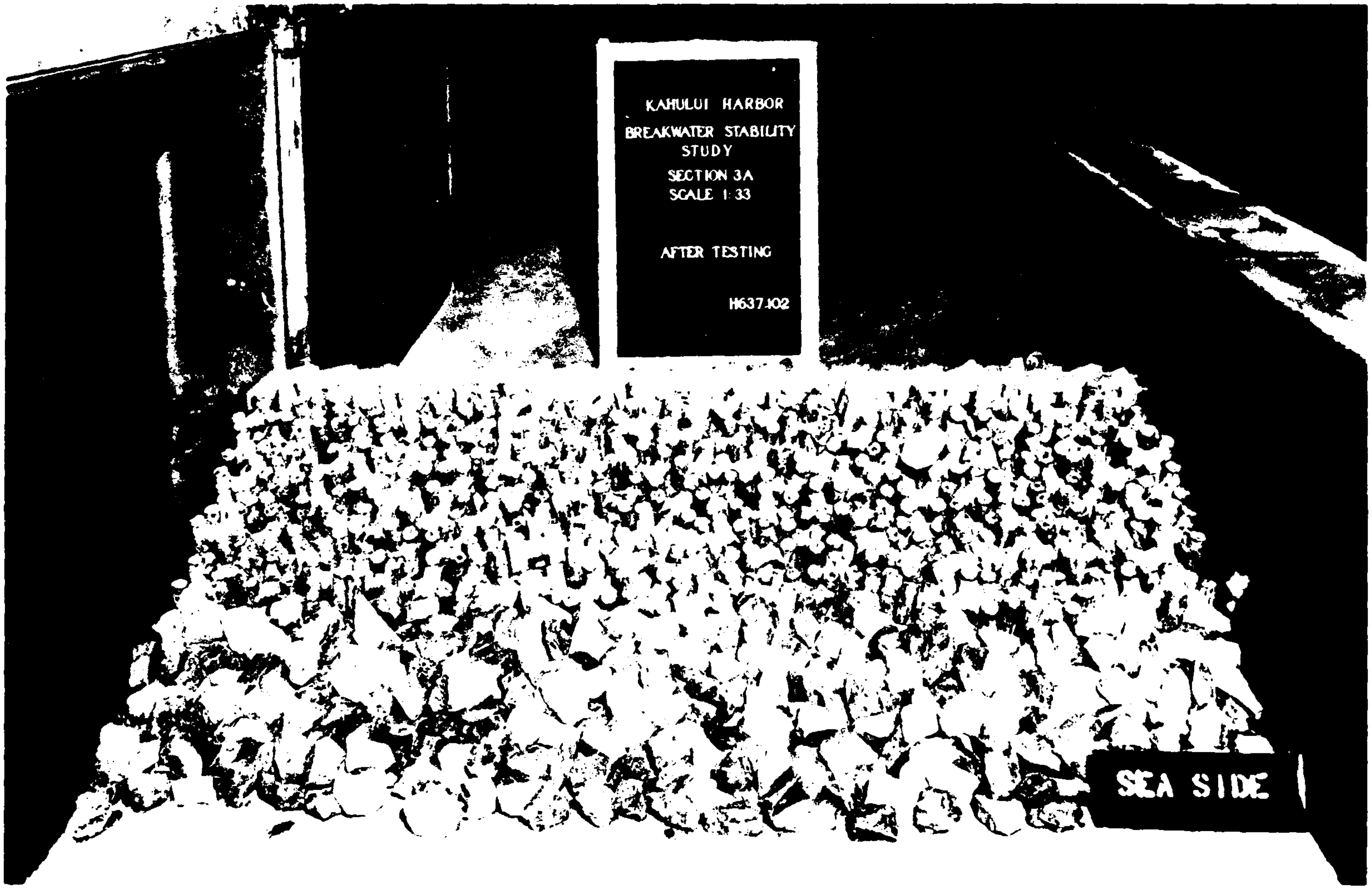


Photo 75. Sea-side view of Plan 2A after testing, 2nd test

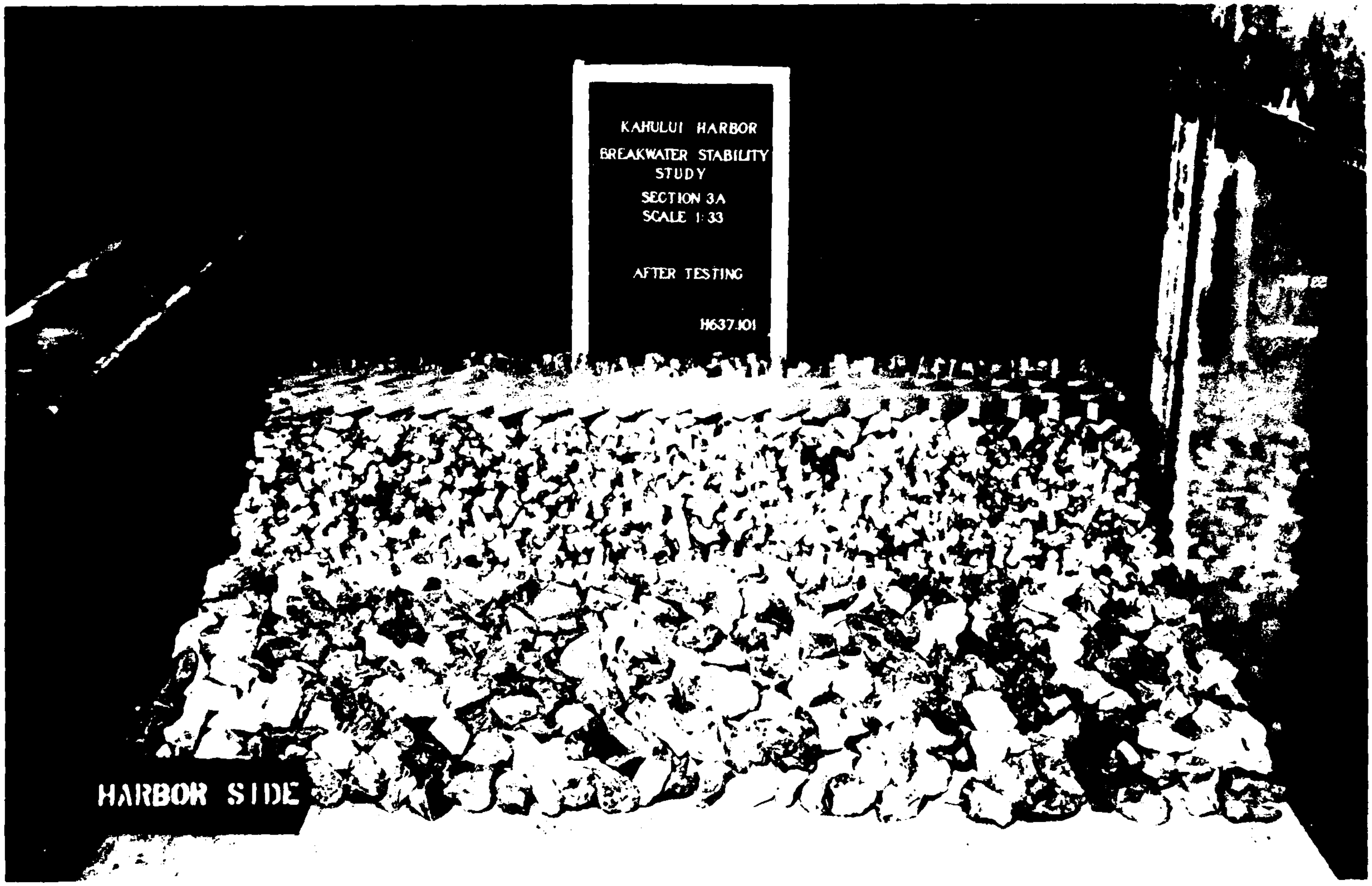


Photo 76. Harbor-side view of Plan 2A after testing, 2nd test

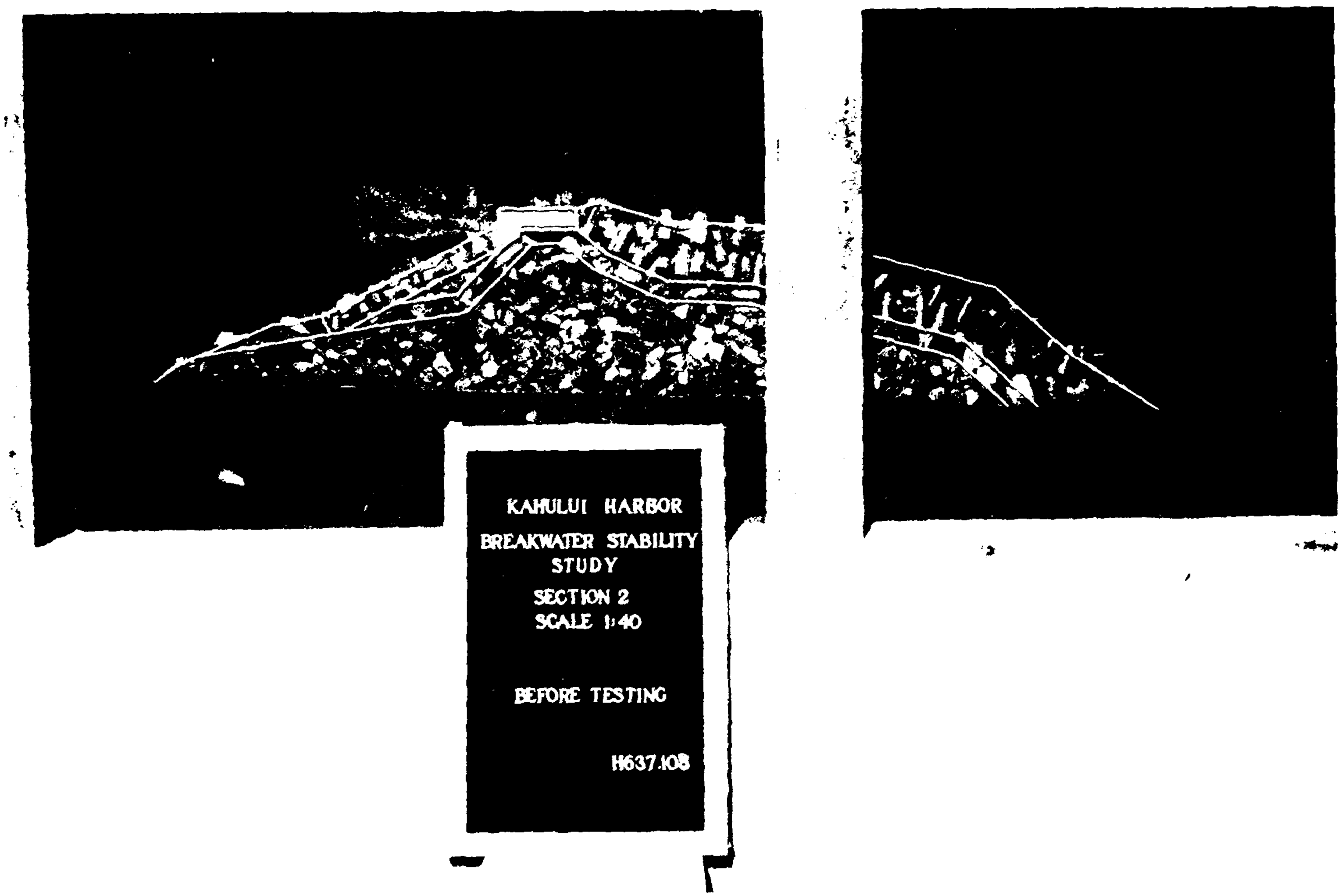


Photo 77. Side view of Plan 3 before testing

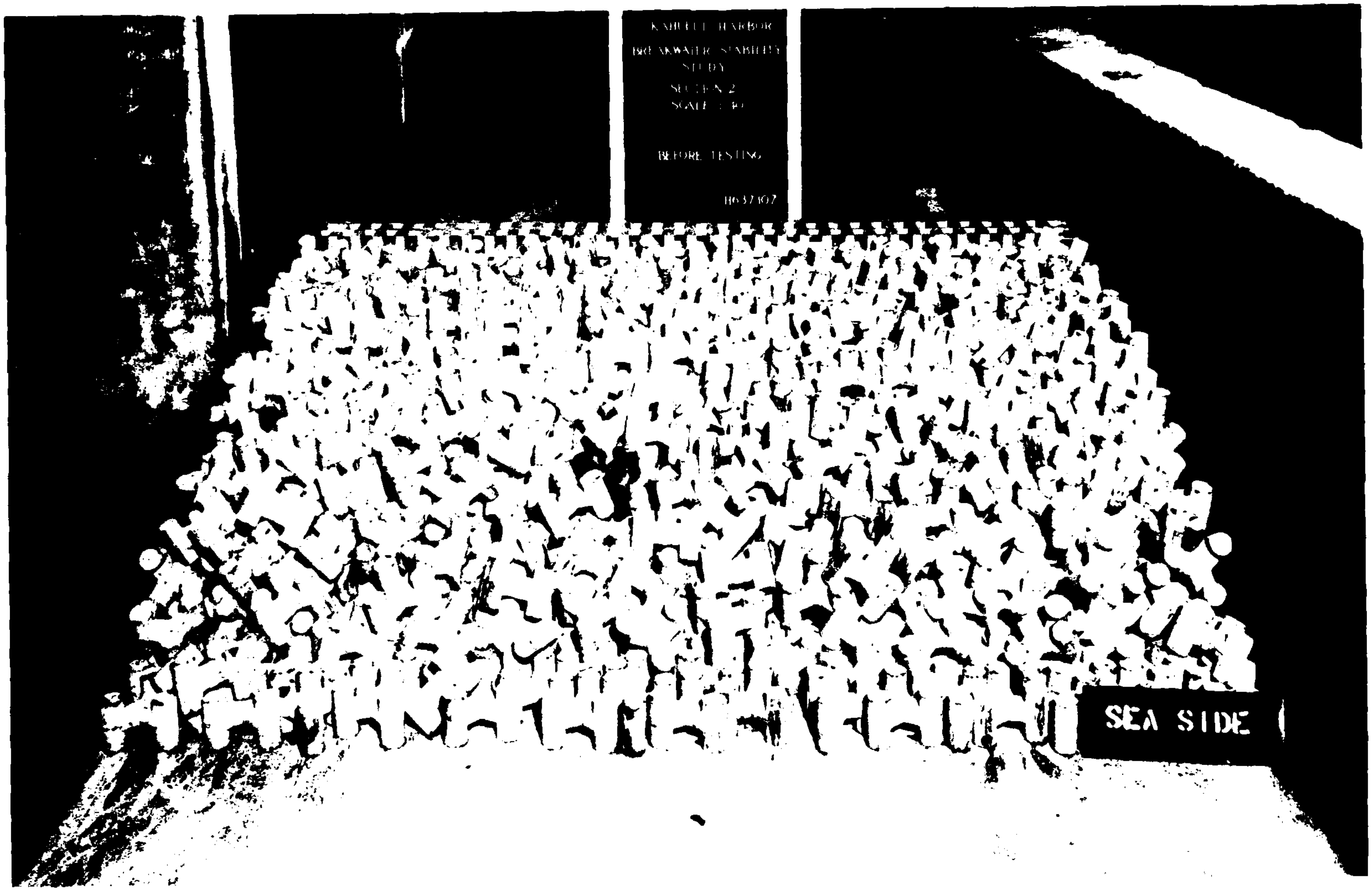


Photo 78. Sea-side view of Plan 3 before testing

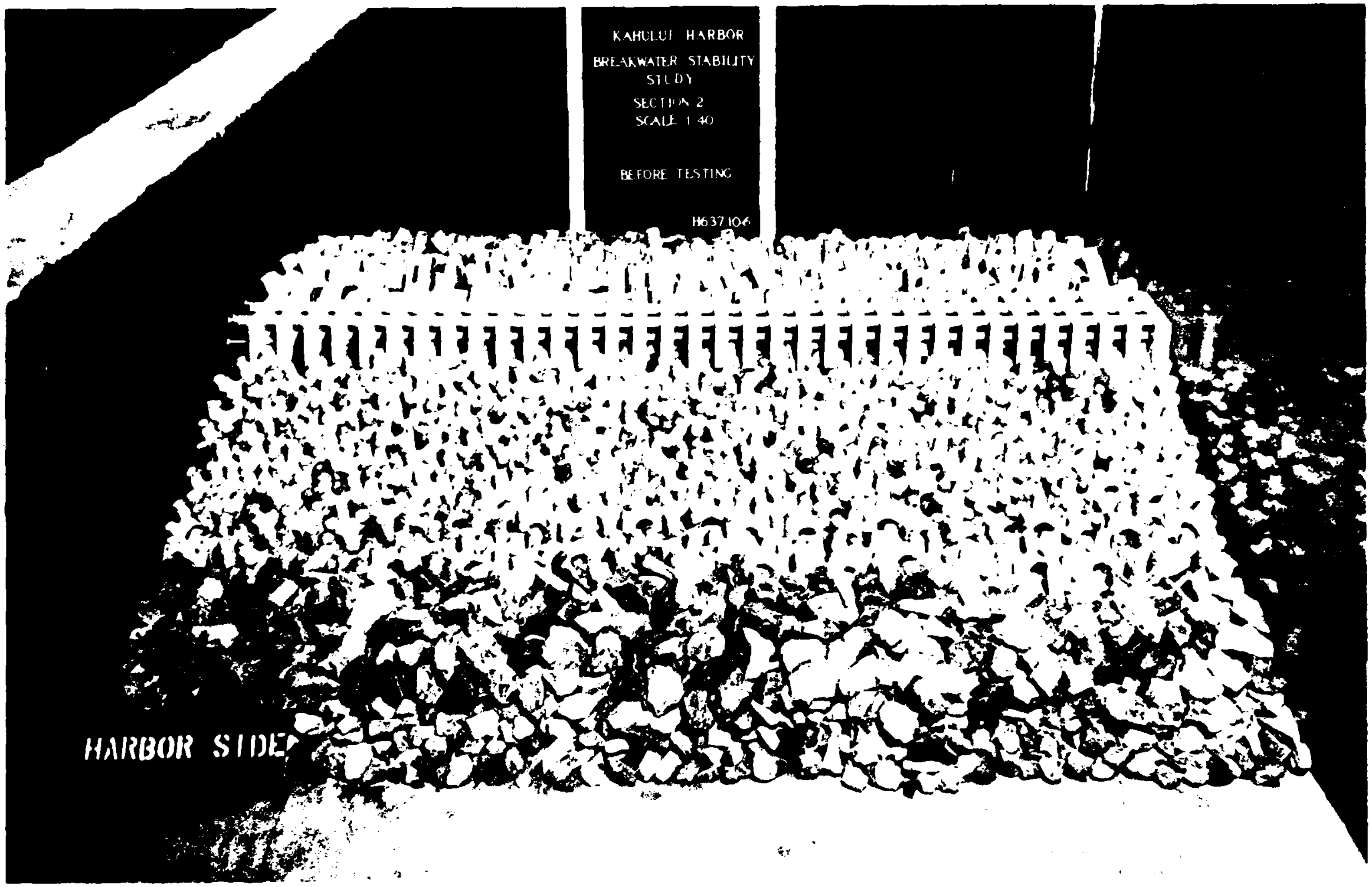


Photo 79. Harbor-side view of Plan 3 before testing

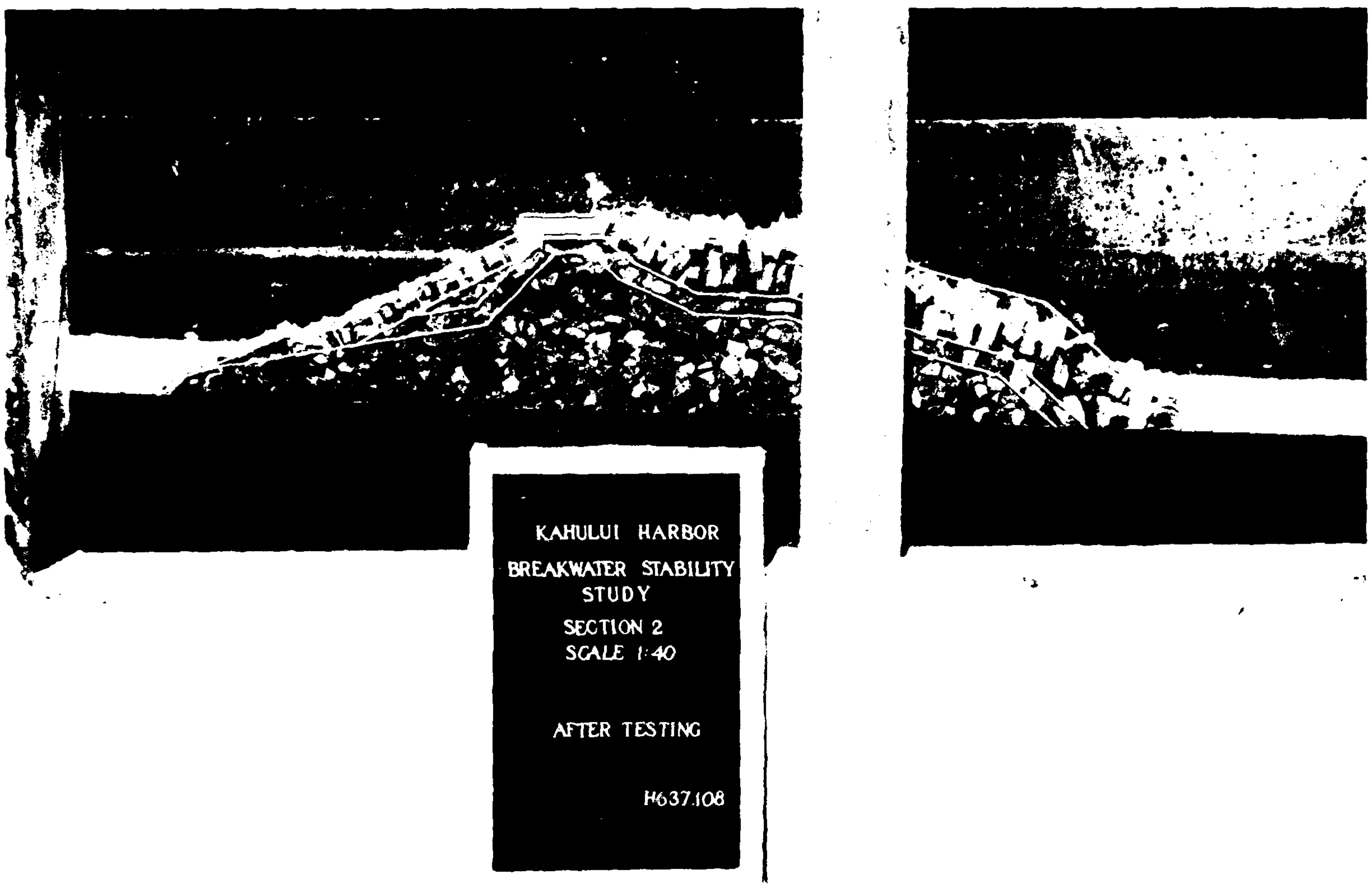


Photo 80. Side view of Plan 3 after testing, 1st test

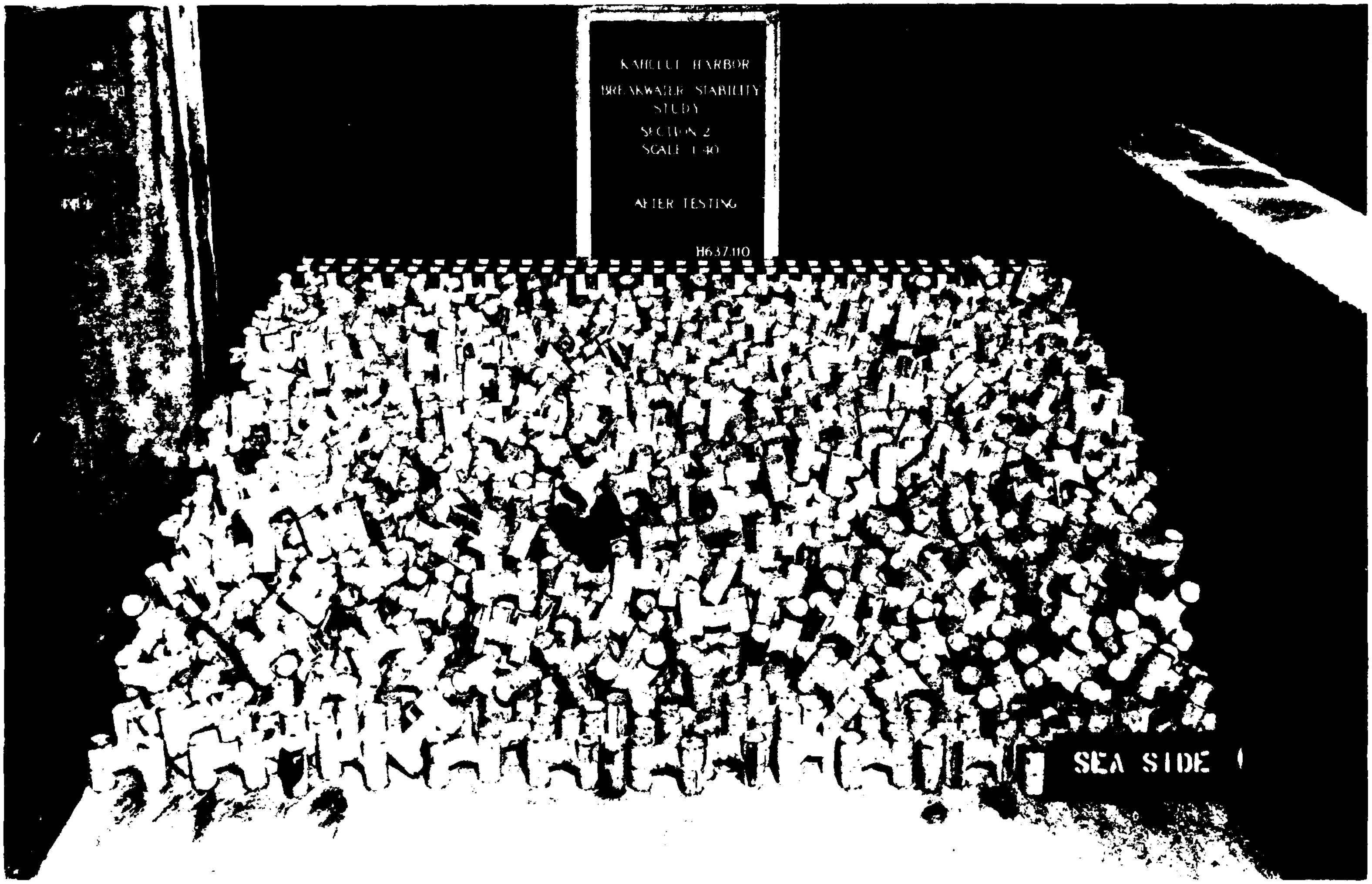


Photo 81. Sea-side view of Plan 3 after testing, 1st test

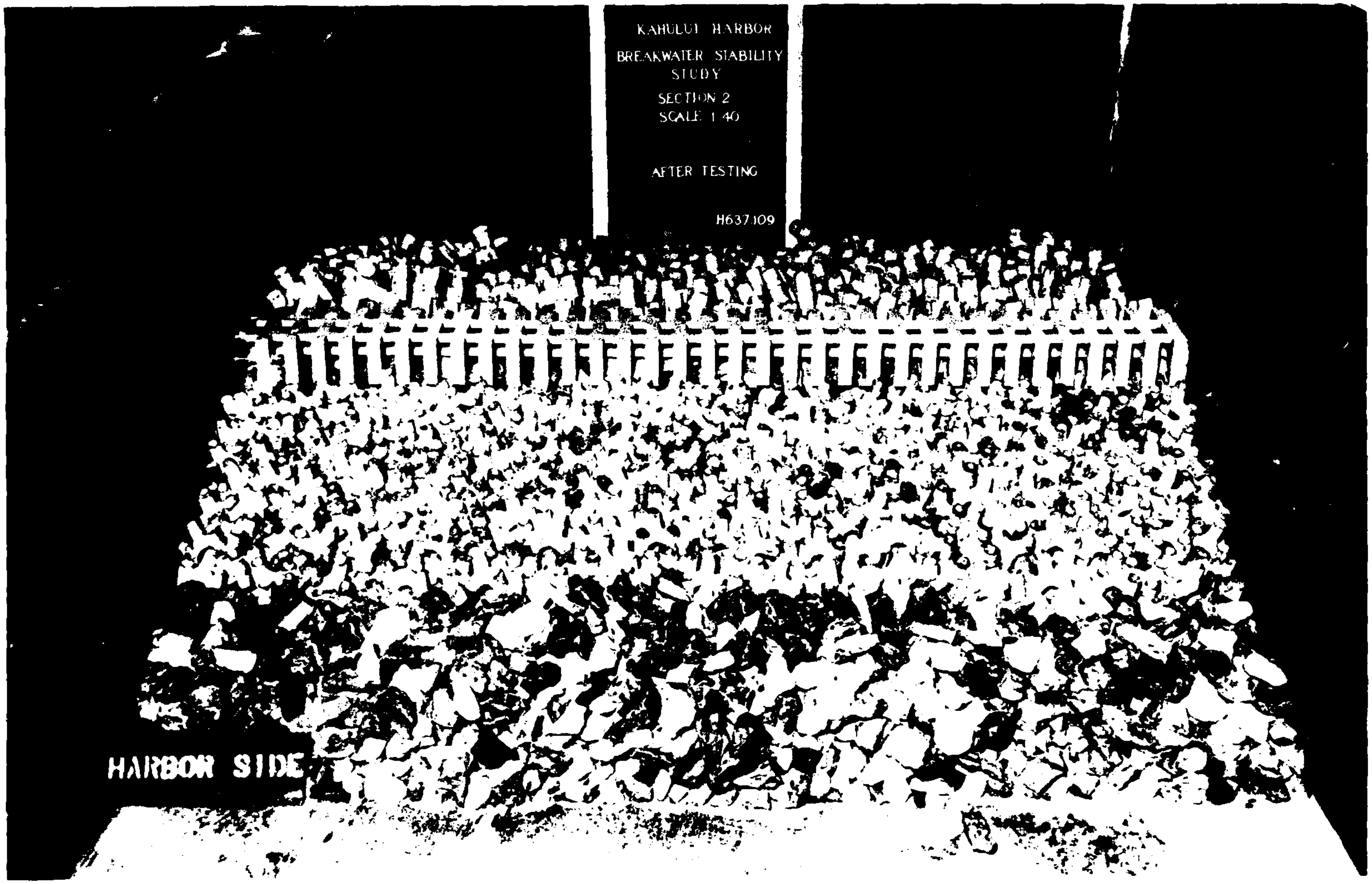


Photo 82. Harbor-side view of Plan 3 after testing, 1st test

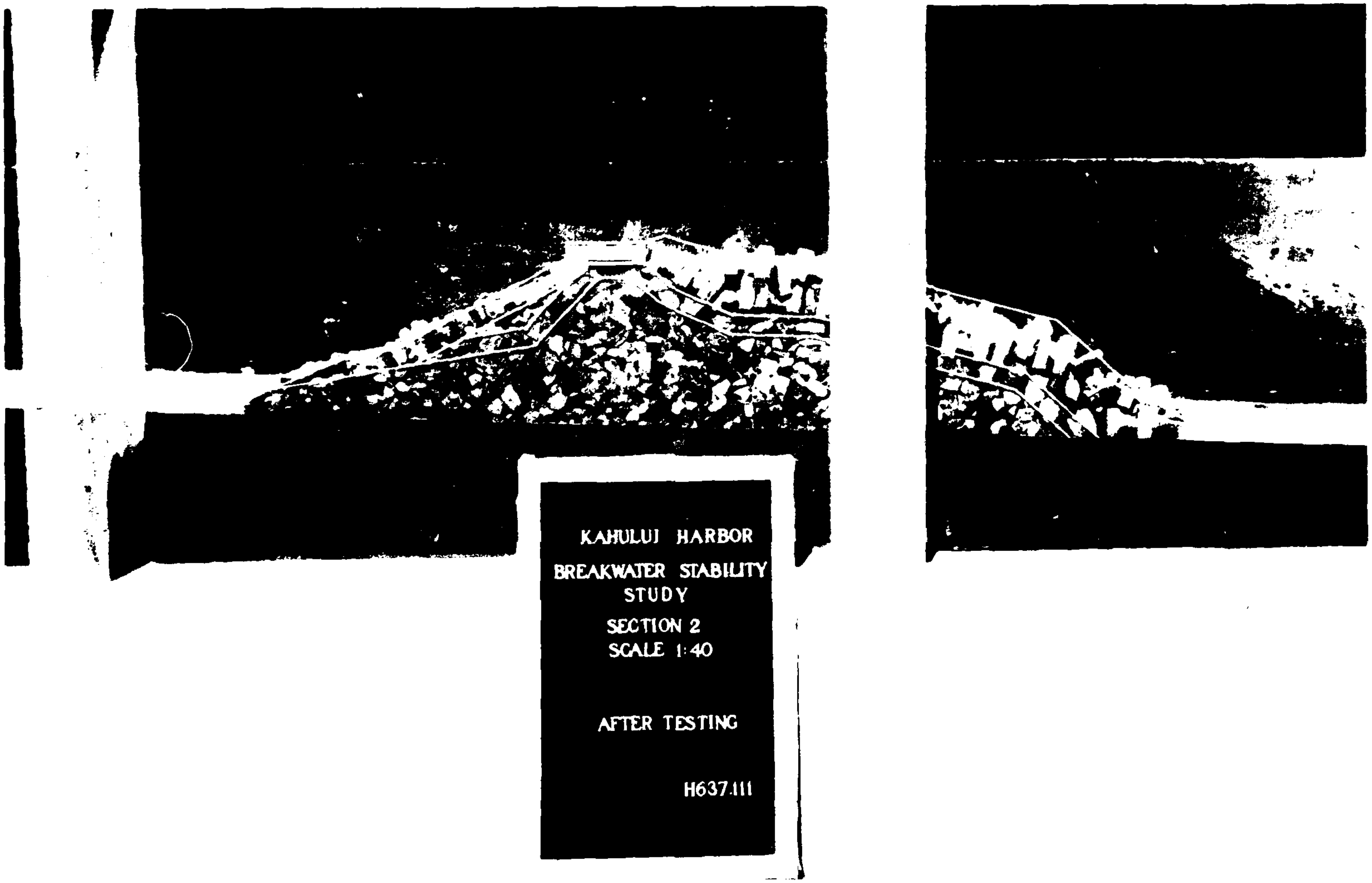


Photo 83. Side view of Plan 3 after testing, 2nd test

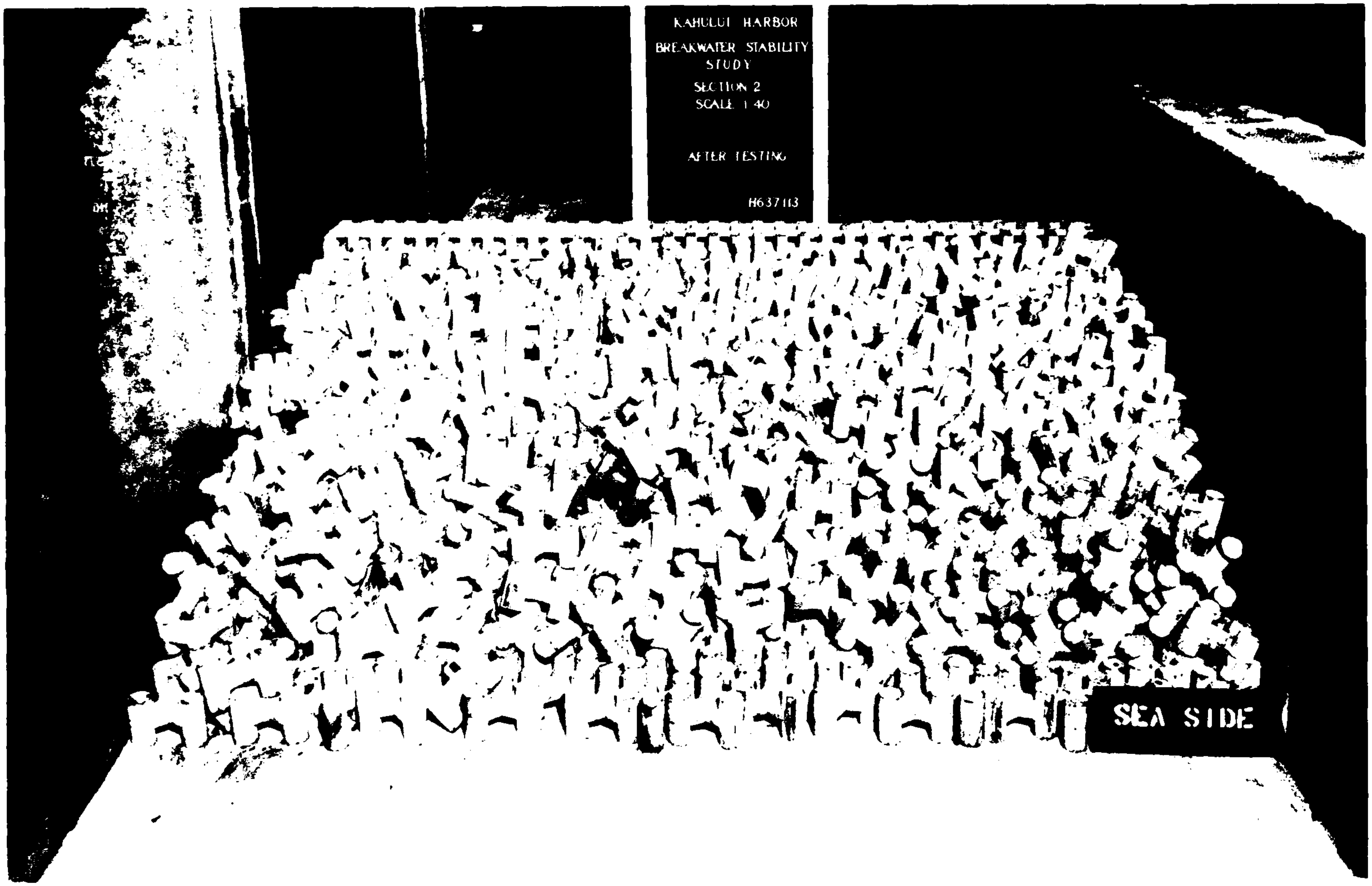


Photo 84. Sea-side view of Plan 3 after testing, 2nd test

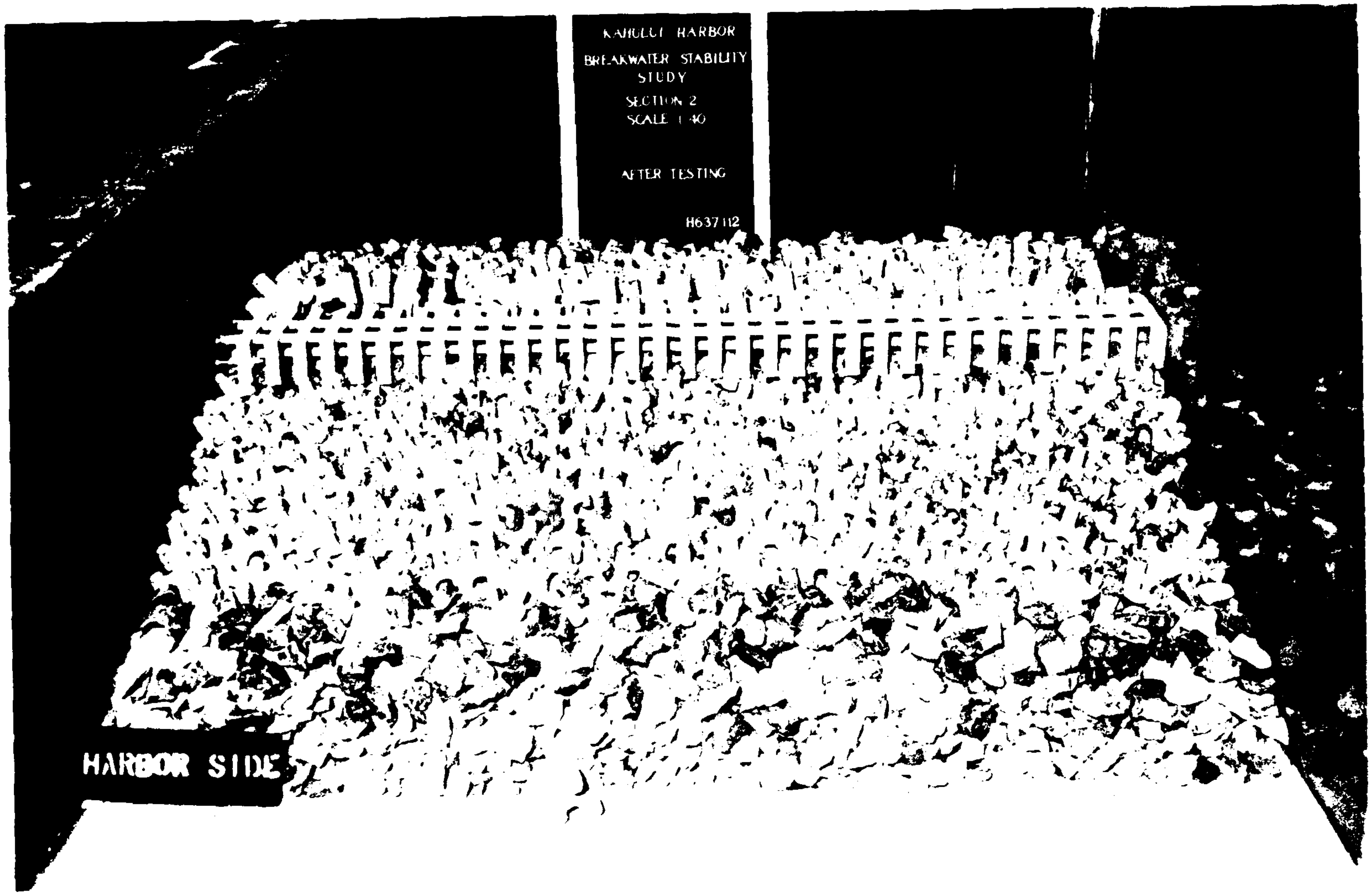


Photo 85. Harbor-side view of Plan 3 after testing, 2nd test

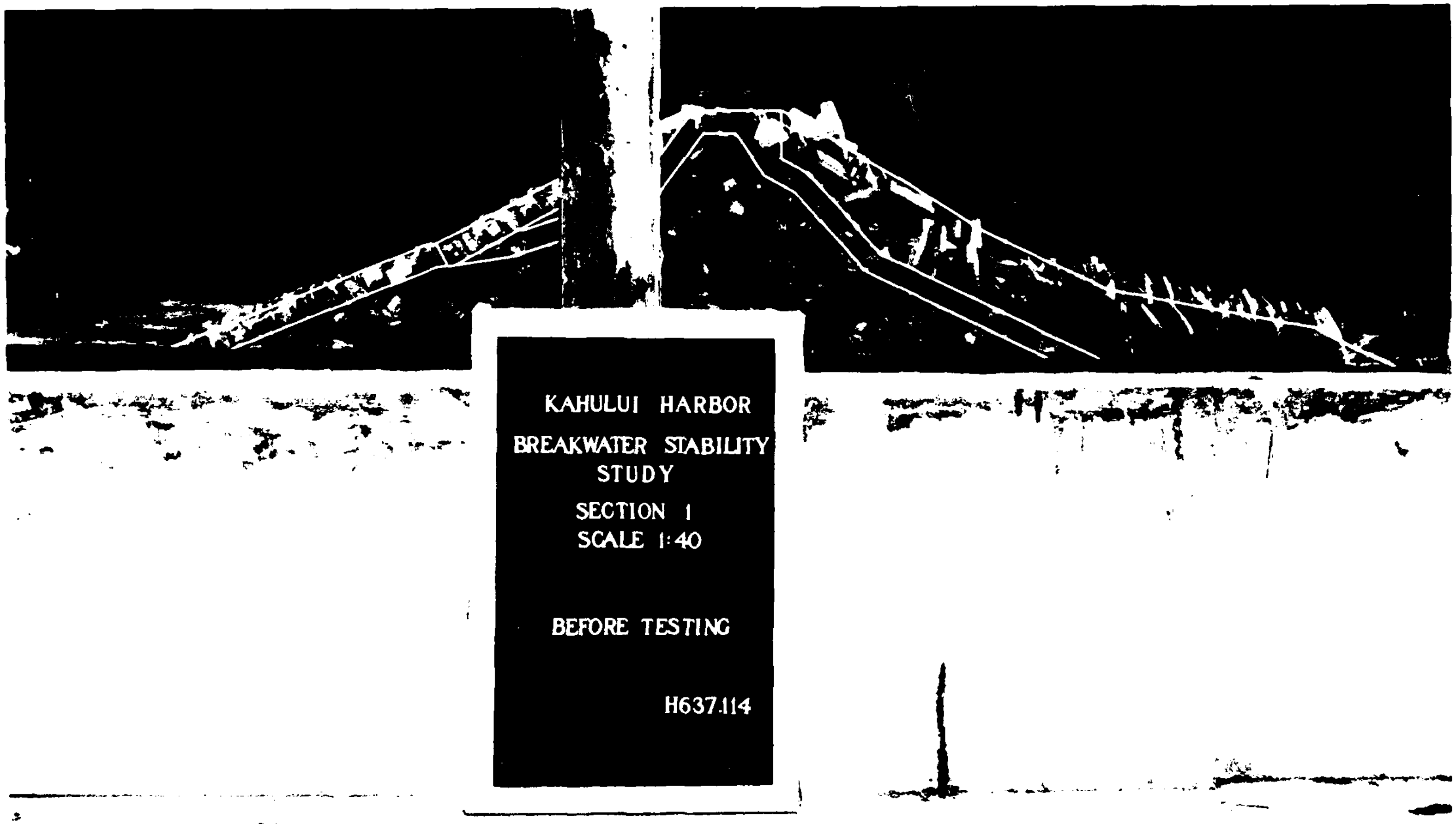


Photo 86. Side view of Plan 4 before testing

Photo 87. Sea-side view of Plan 4 before testing

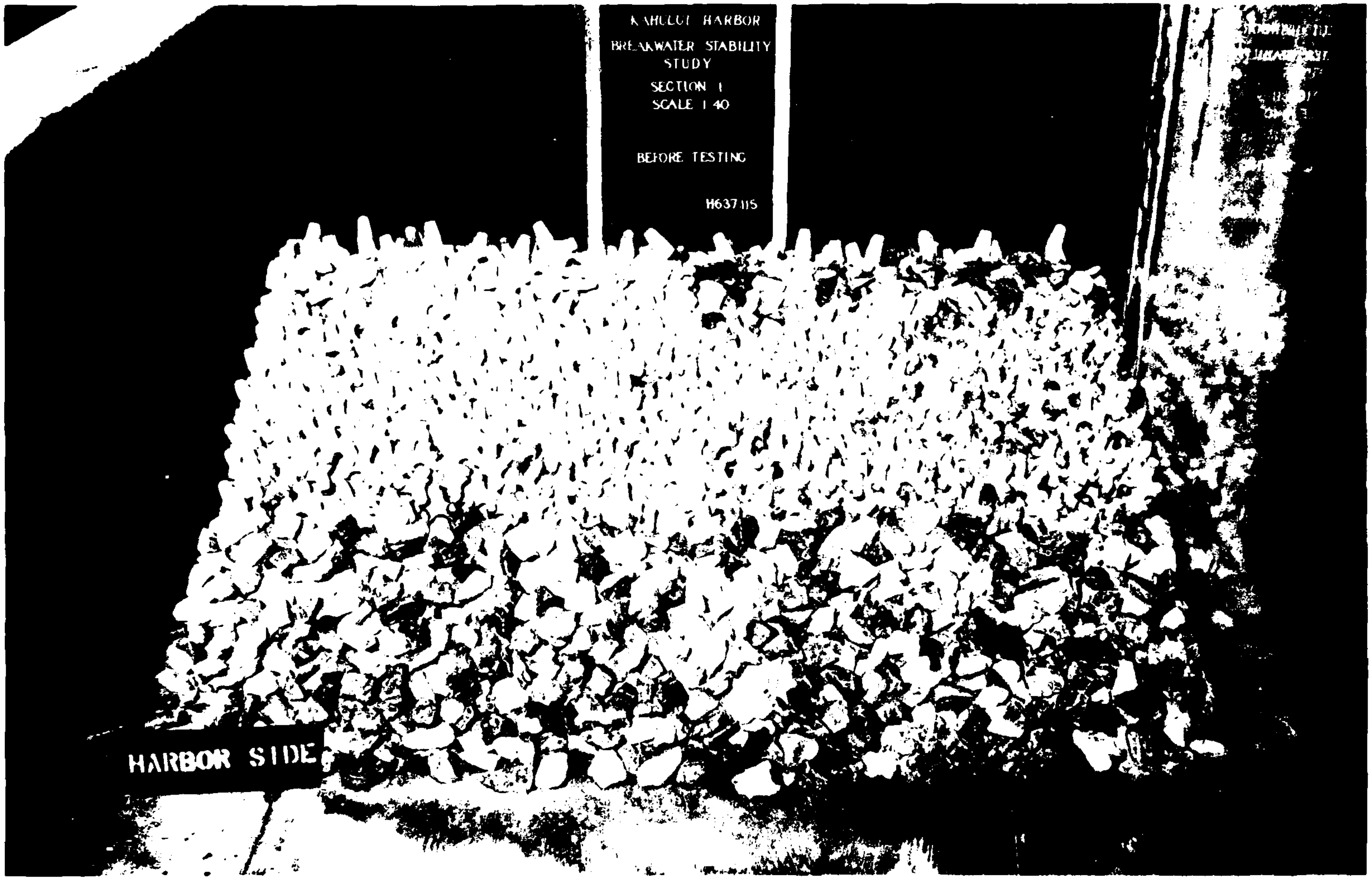


Photo 88. Harbor-side view of Plan 4 before testing

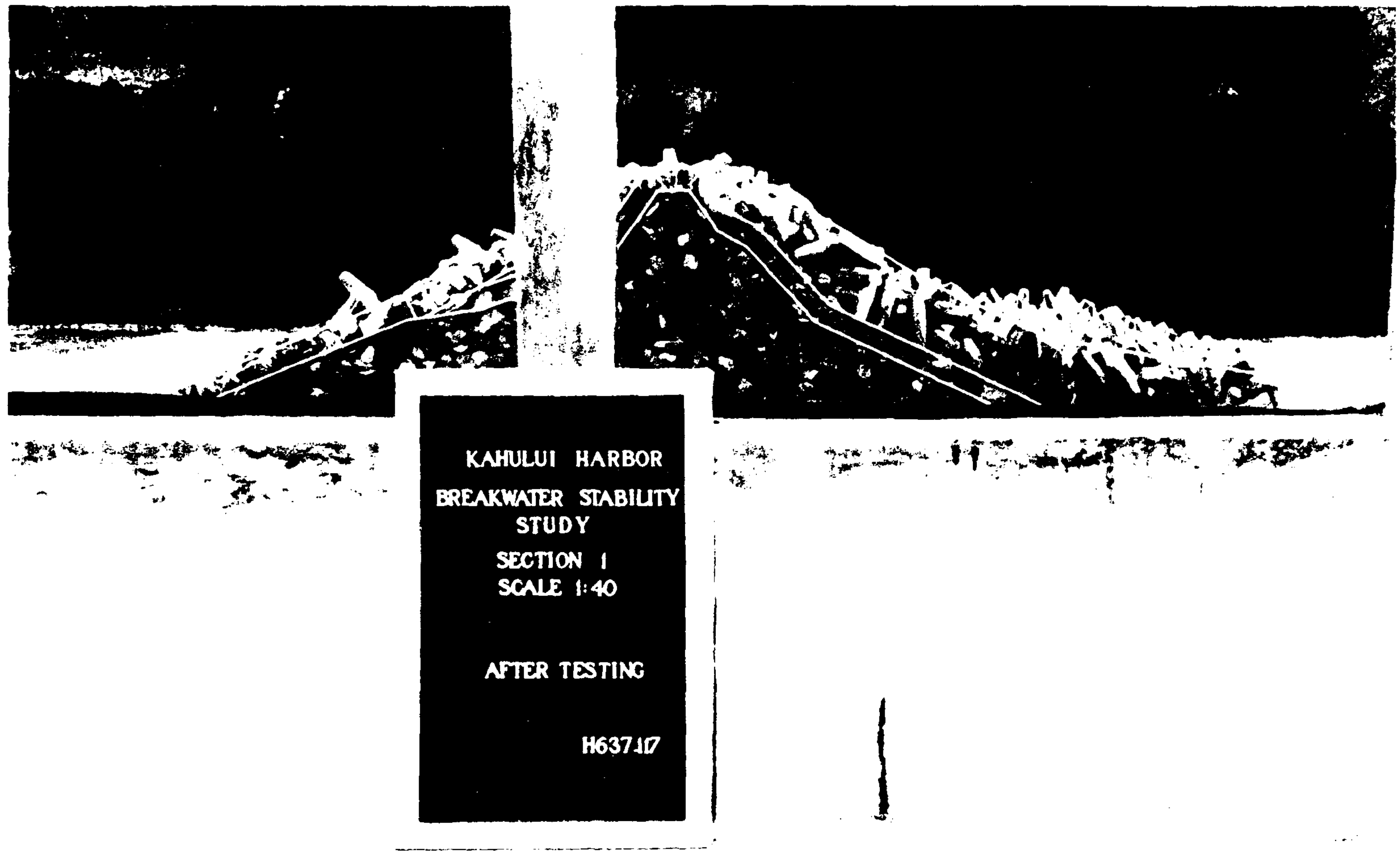


Photo 89. Side view of Plan 4 after testing

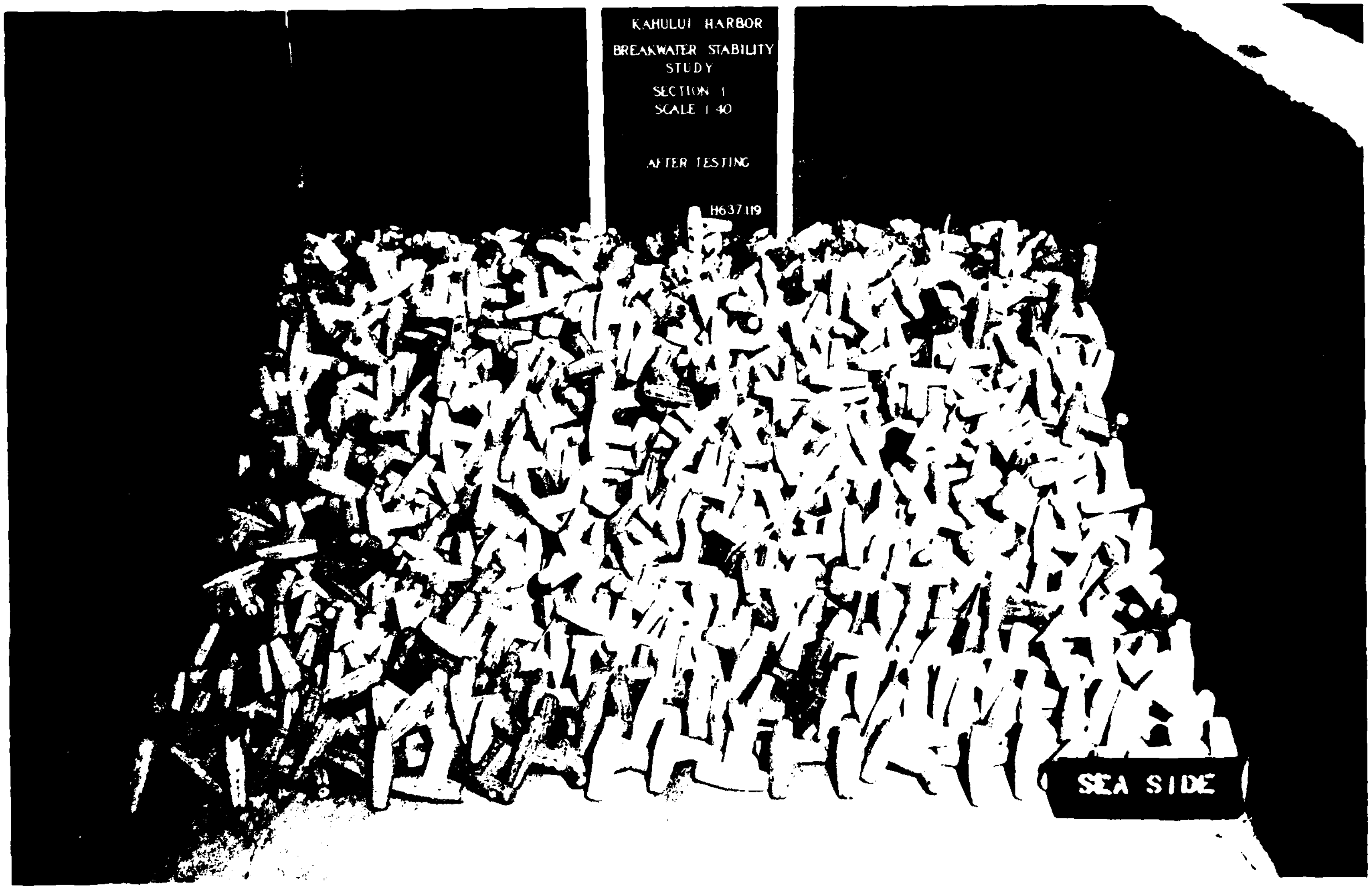


Photo 90. Sea-side view of Plan 4 after testing

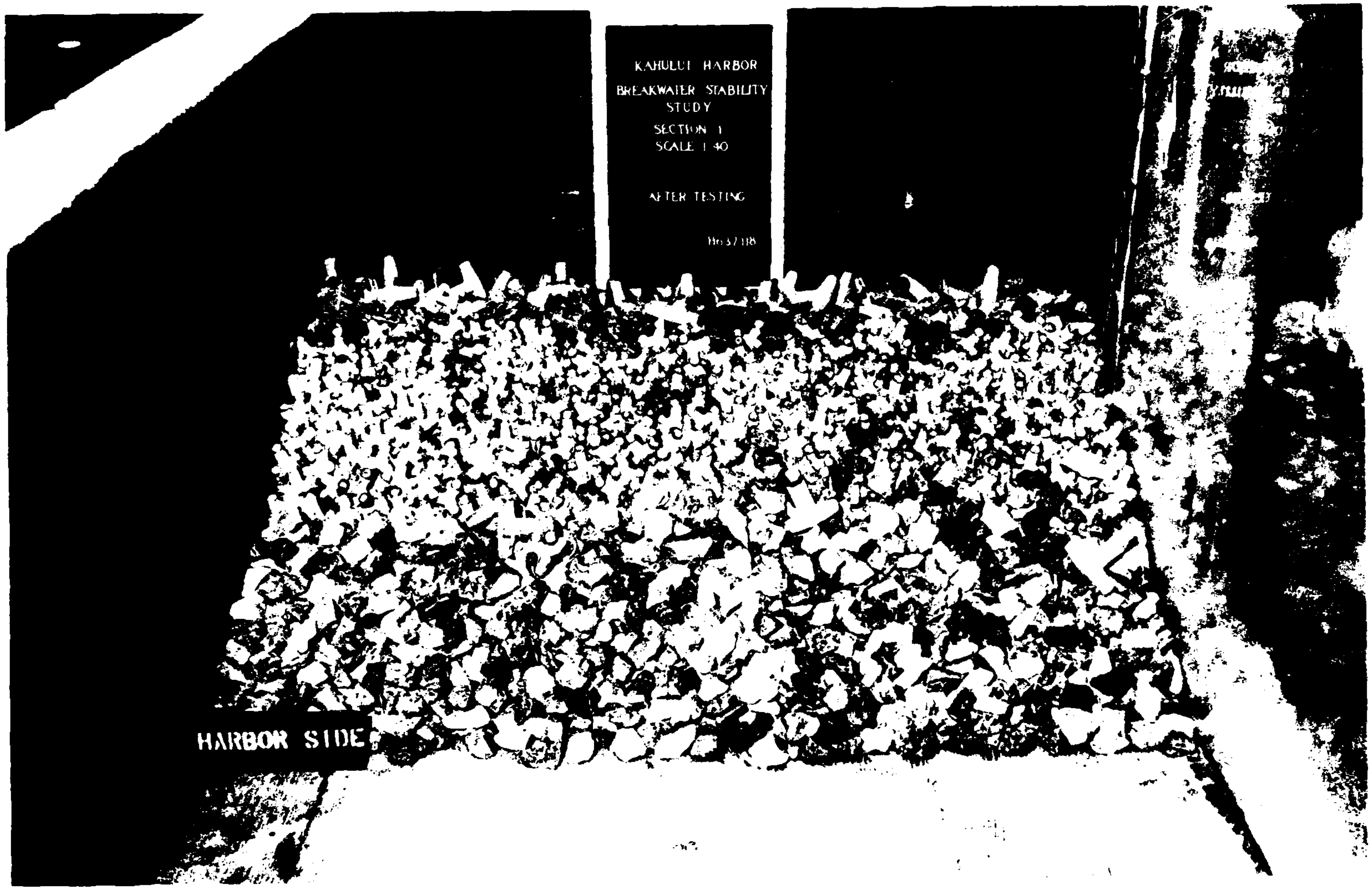


Photo 91. Harbor-side view of Plan 4 after testing

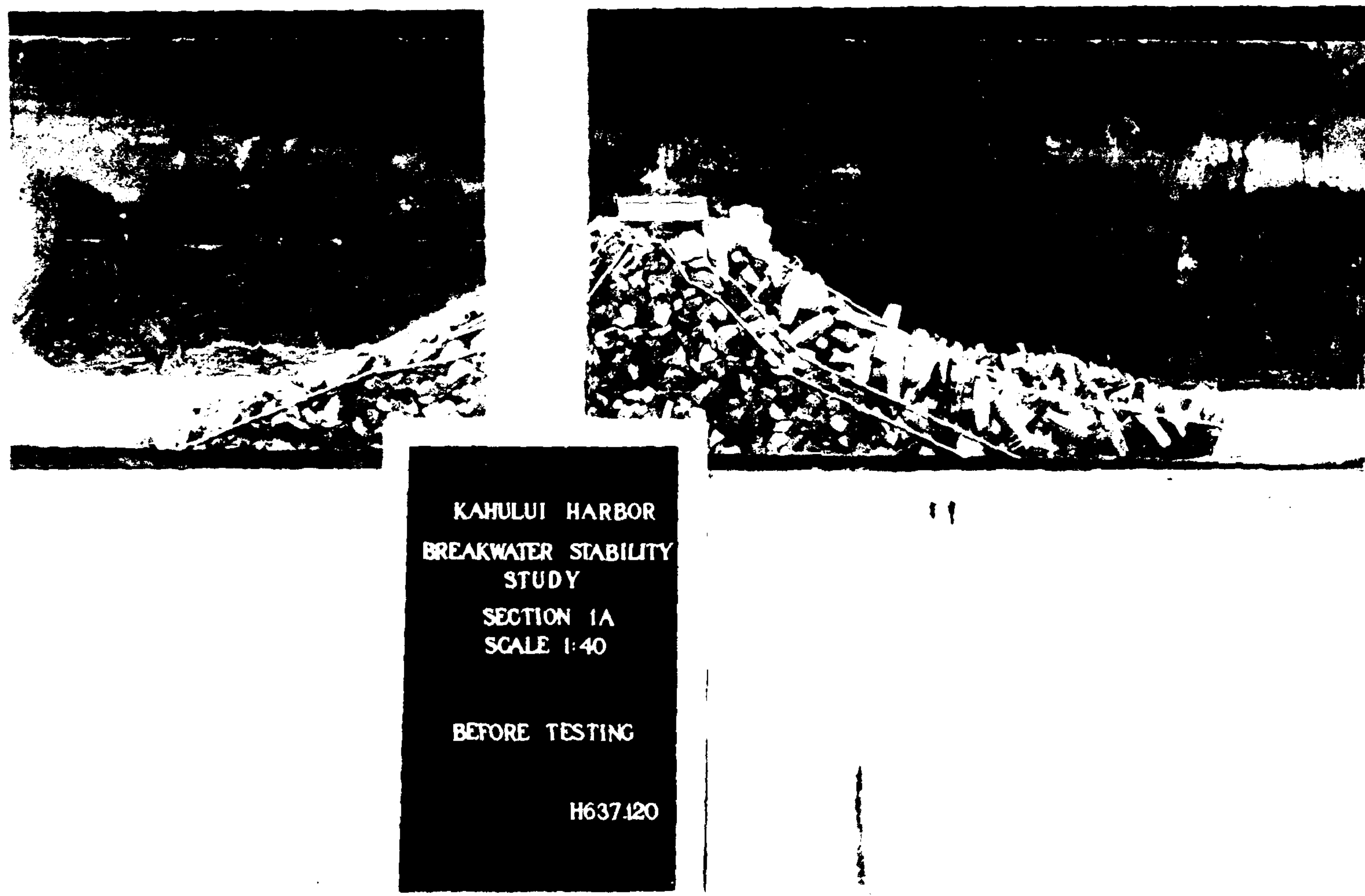


Photo 92. Side view of Plan 4A before testing

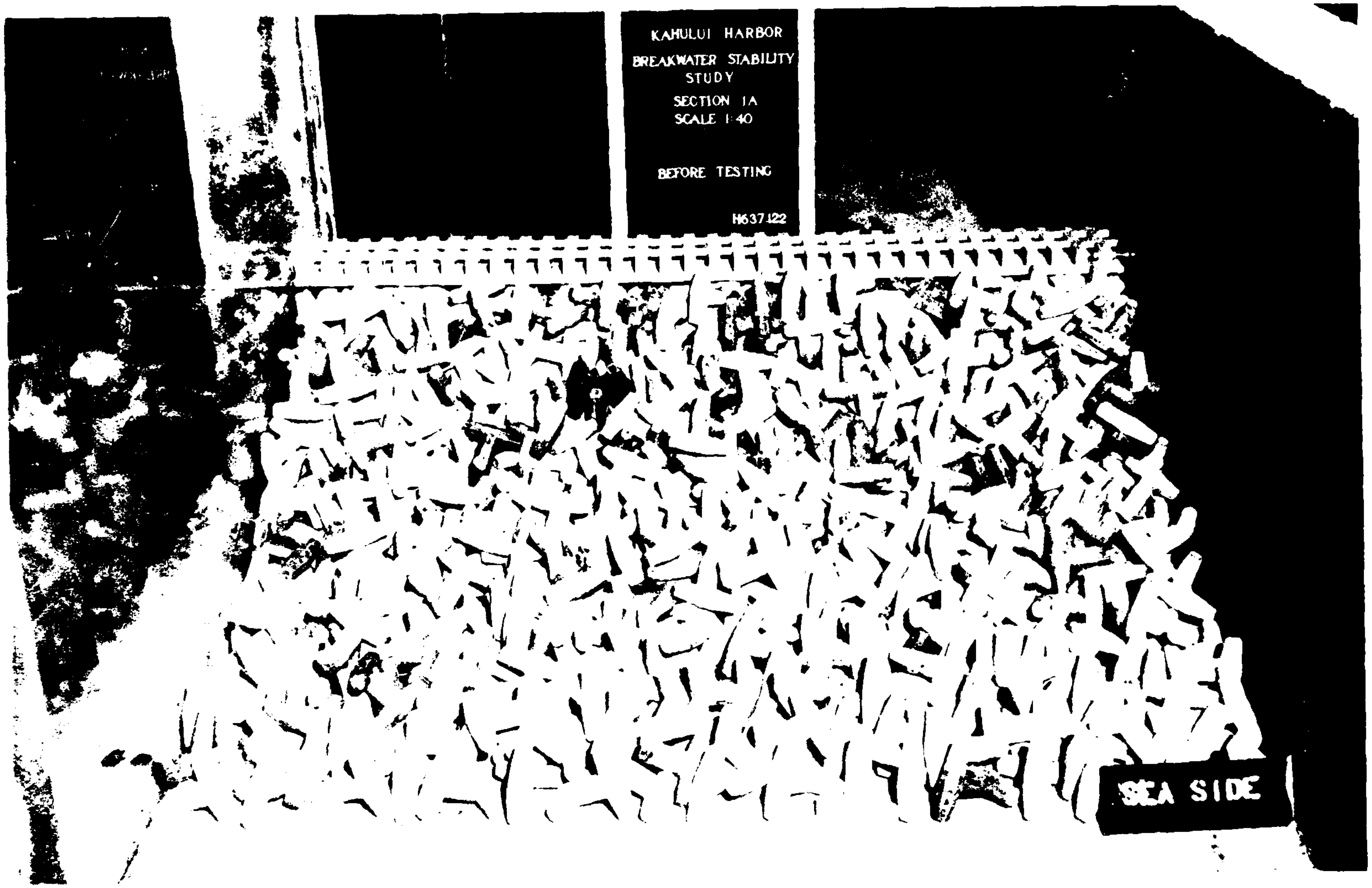


Photo 93. Sea-side view of Plan 4A before testing

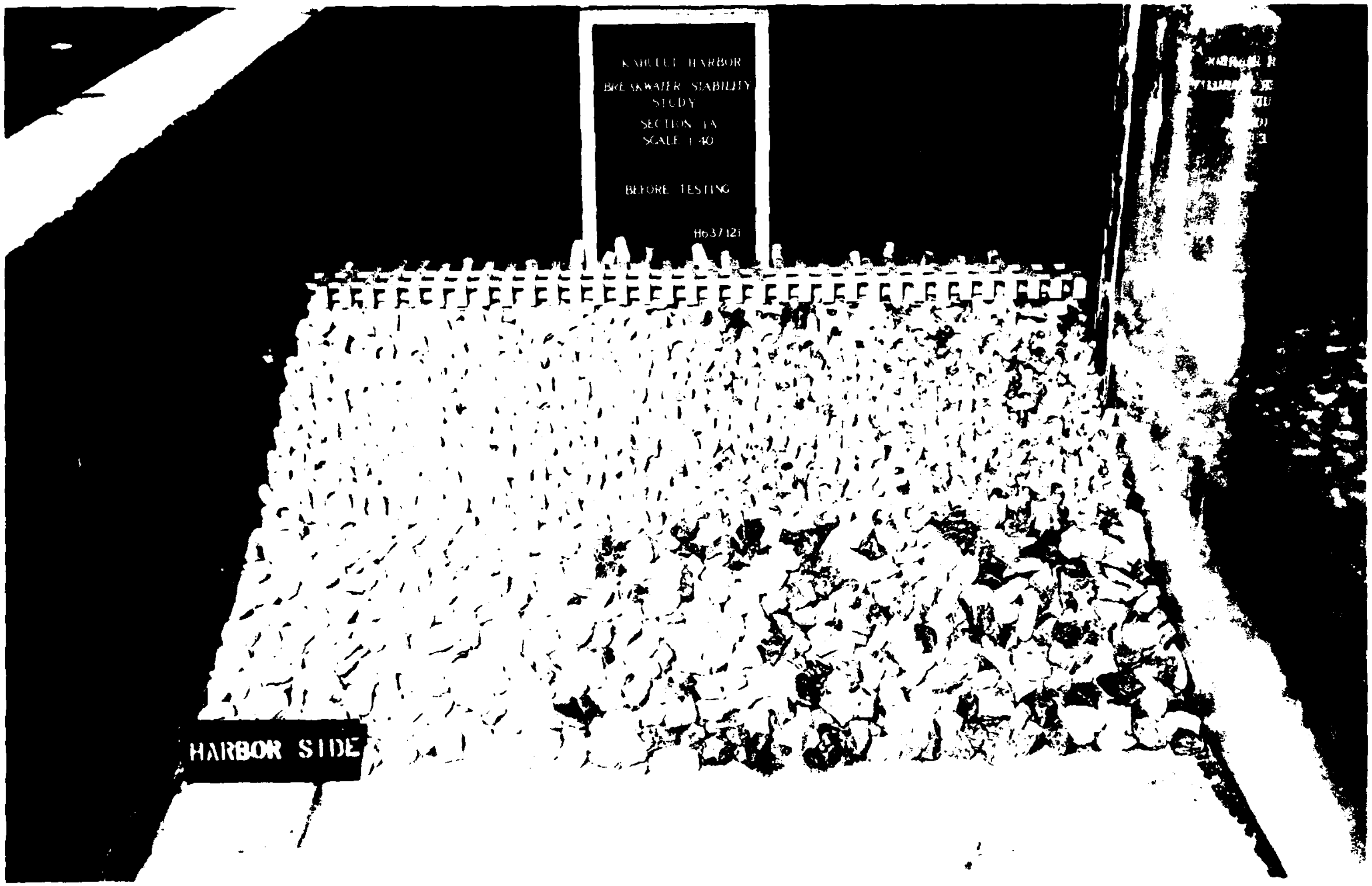


Photo 94. Harbor-side view of Plan 4A before testing

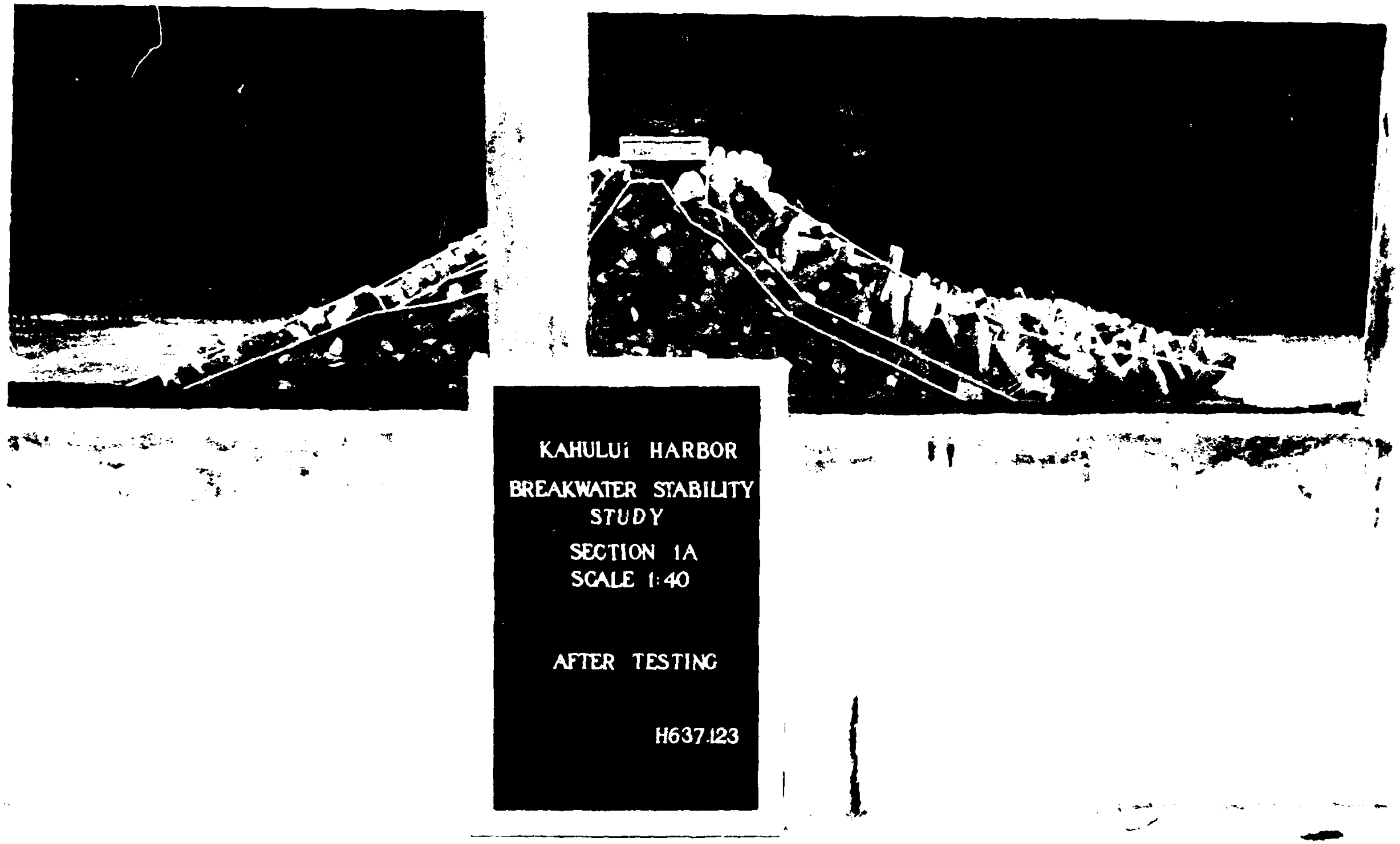


Photo 95. Side view of Plan 4A after testing, 1st test

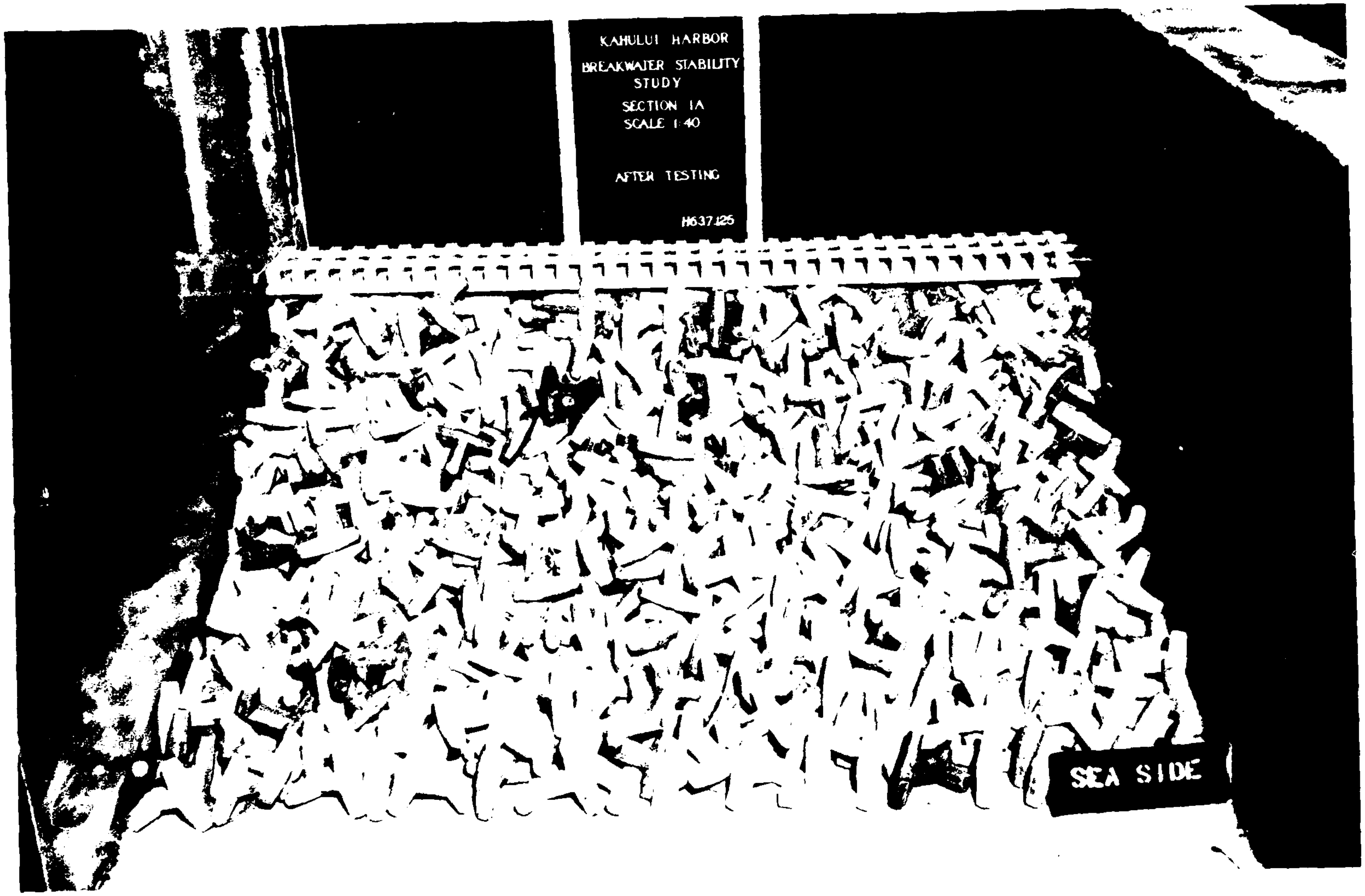


Photo 96. Sea-side view of Plan 4A after testing, 1st test

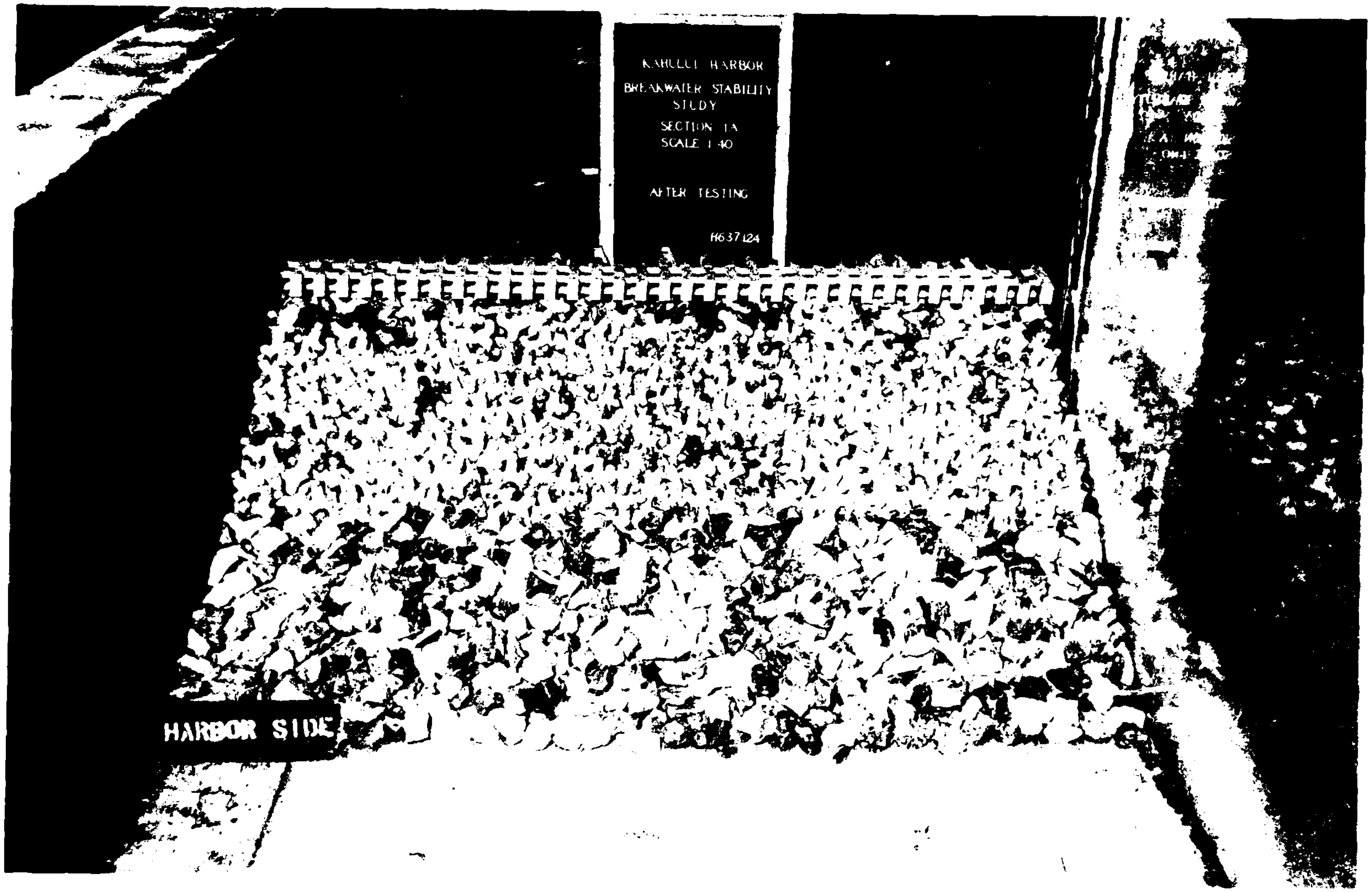


Photo 97. Harbor-side view of Plan 4A after testing, 1st test

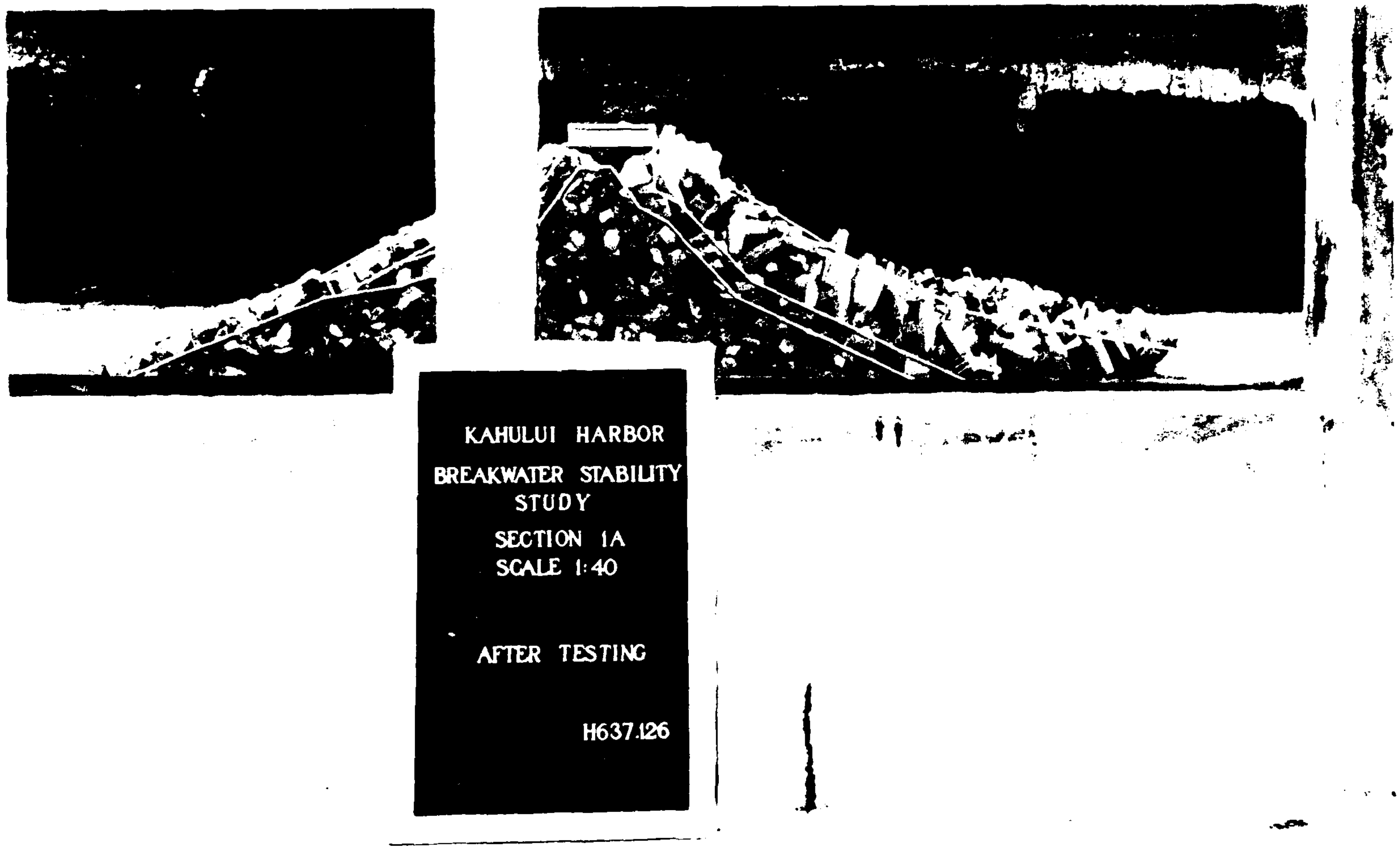


Photo 98. Side view of Plan 4A after testing, 2nd test

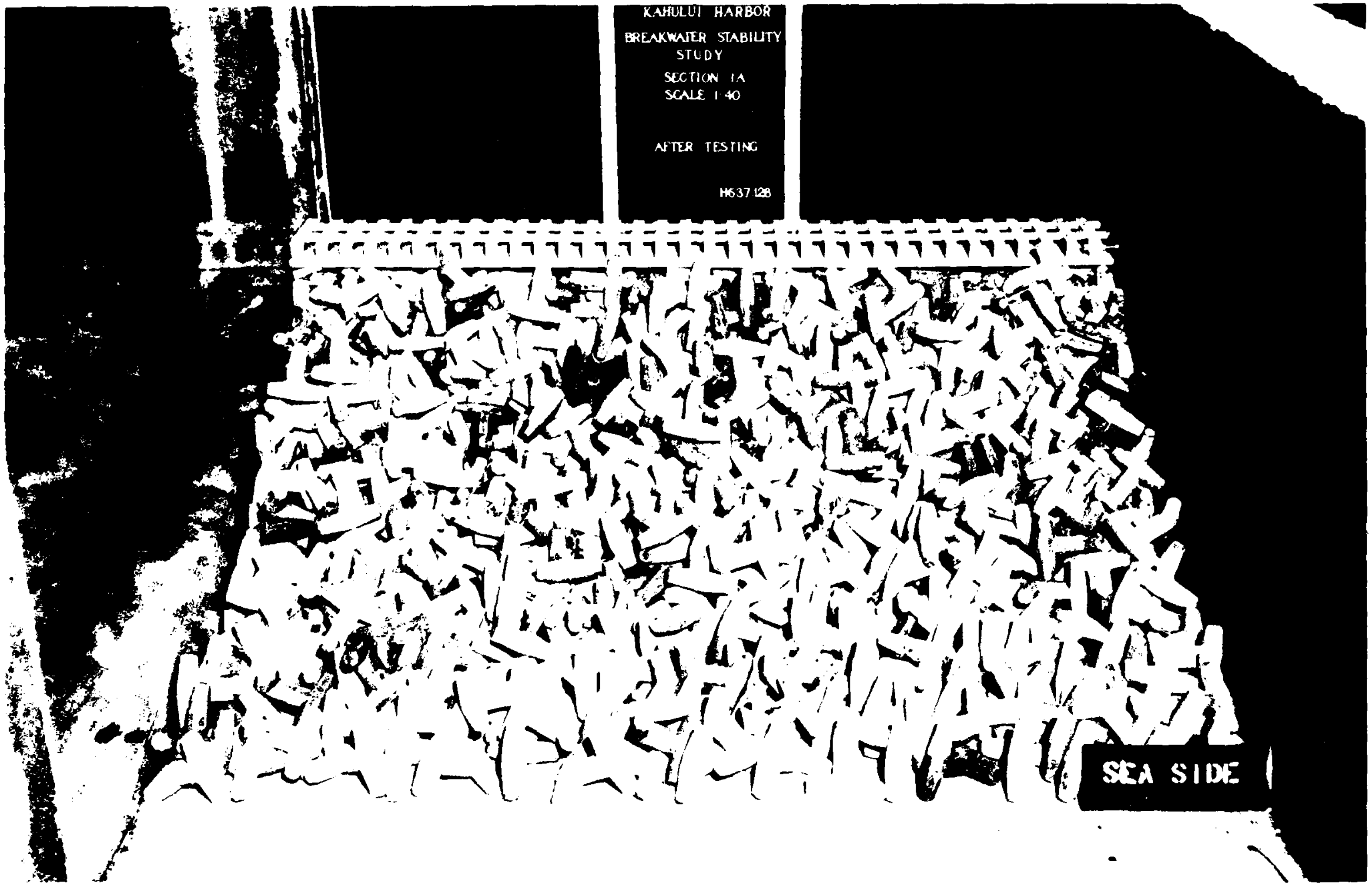


Photo 99. Sea-side view of Plan 4A after testing, 2nd test

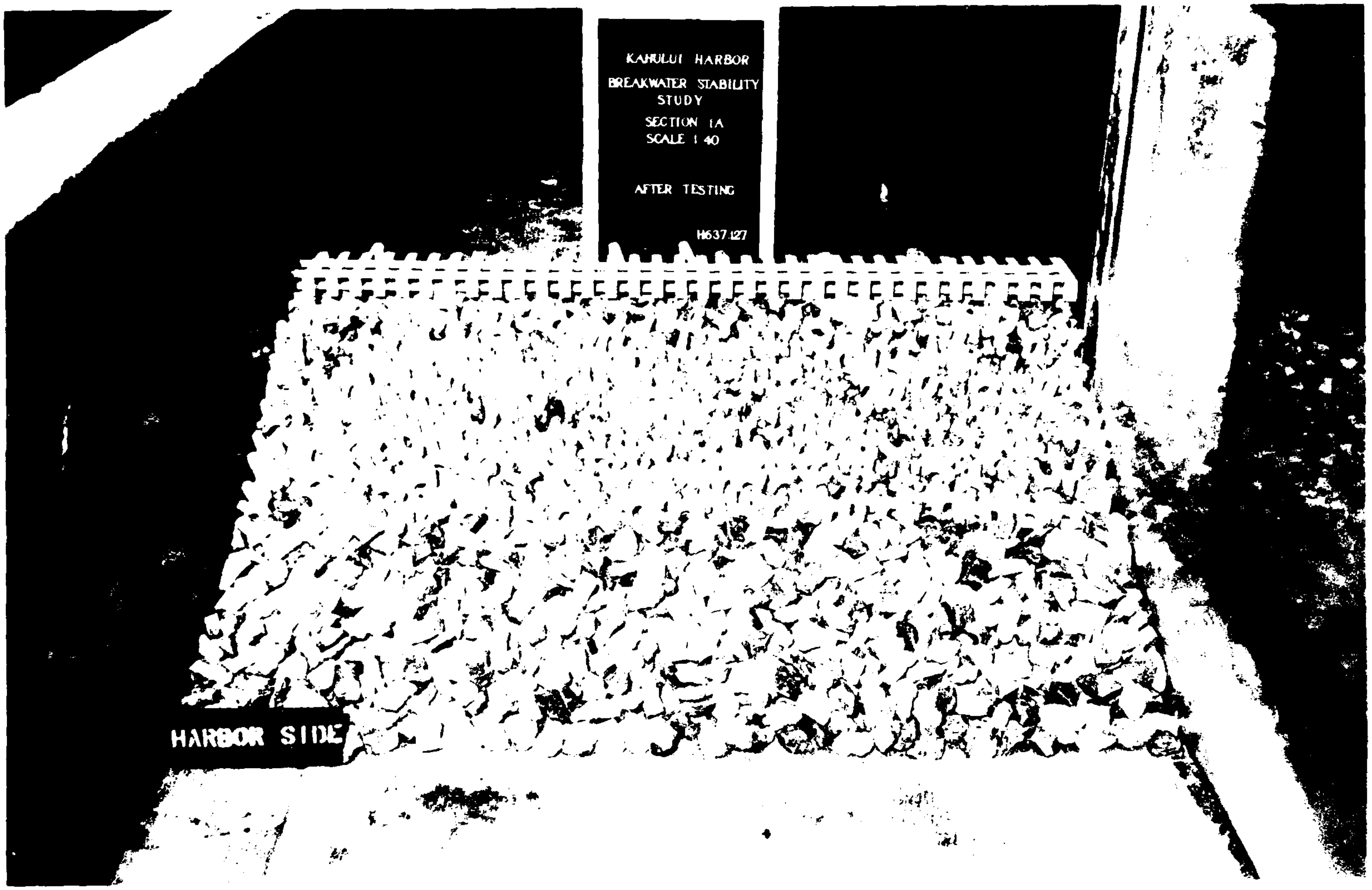
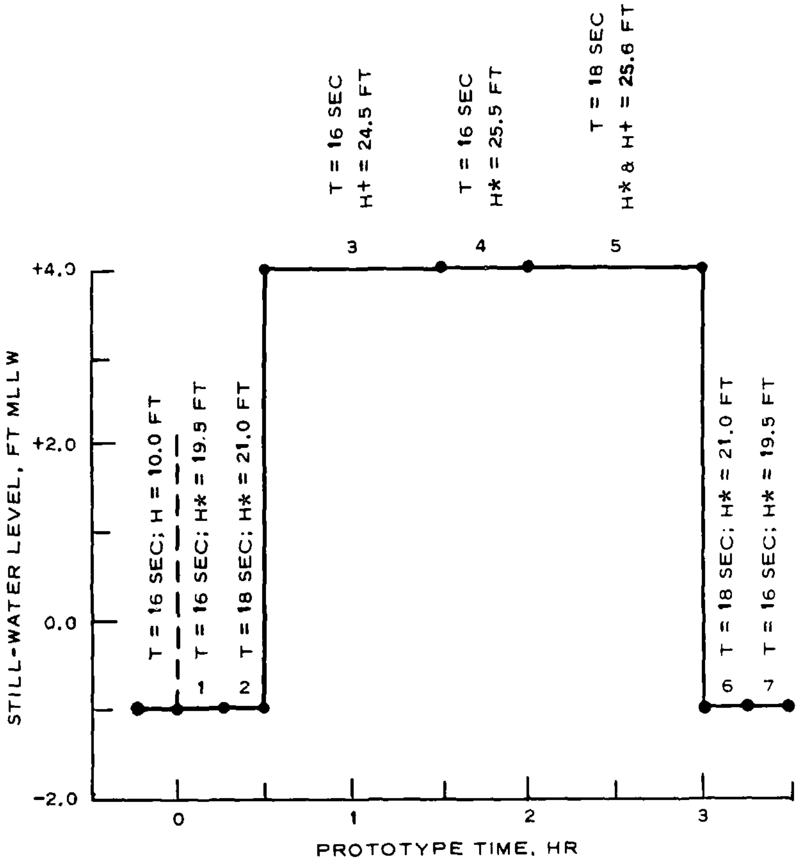


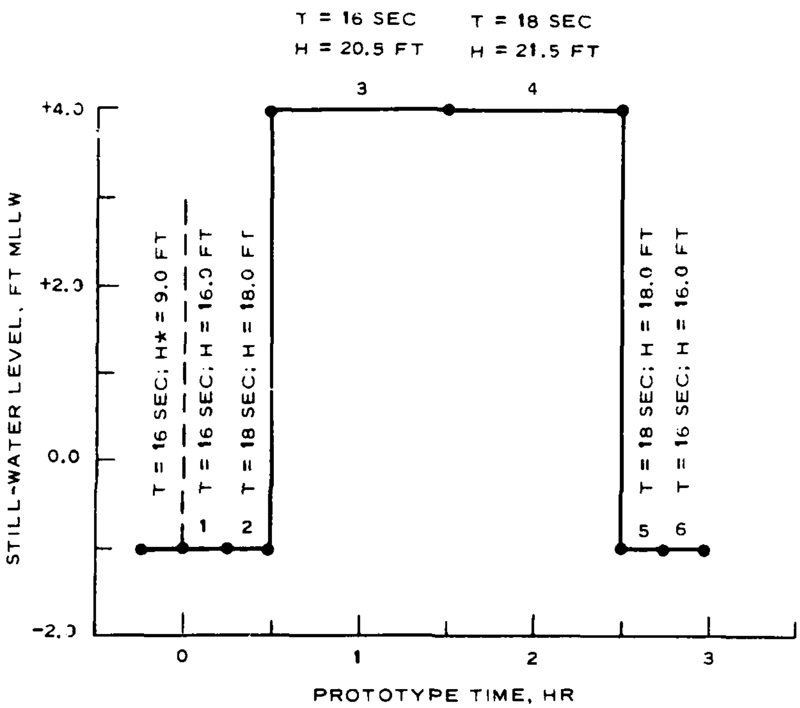
Photo 100. Harbor-side view of Plan 4A after testing, 2nd test



LEGEND

T	WAVE PERIOD
H	SHAKEDOWN WAVE HEIGHT
H+	WORST BREAKING WAVE HEIGHT FOR HARBOR-SIDE STABILITY
H*	WORST BREAKING WAVE HEIGHT FOR SEA-SIDE STABILITY
1-7	HYDROGRAPH STEPS

HYDROGRAPH A

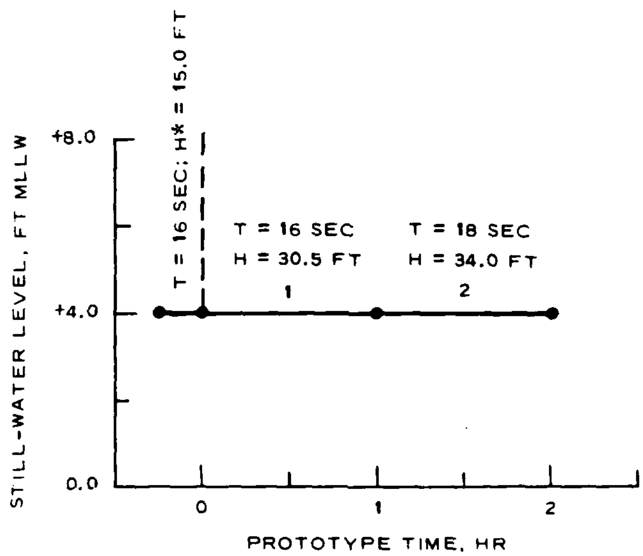


LEGEND

T WAVE PERIOD
H* SHAKEDOWN WAVE HEIGHT
H WORST BREAKING WAVE HEIGHT
1–6 HYDROGRAPH STEPS

HYDROGRAPH B

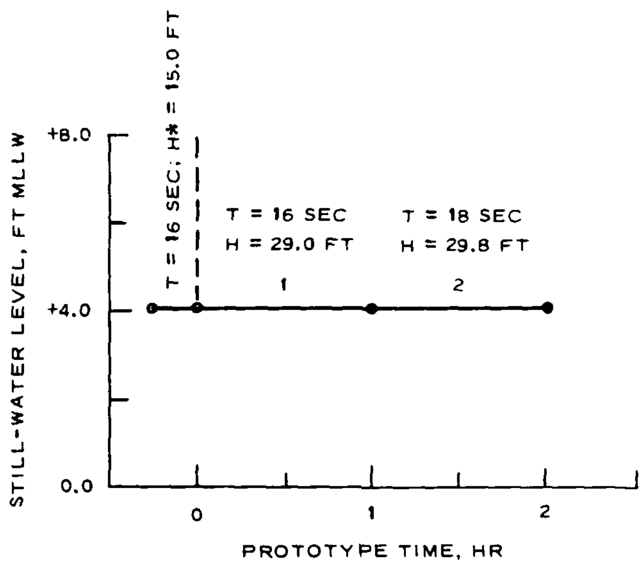
PLATE 2



LEGEND

T	WAVE PERIOD
H*	SHAKEDOWN WAVE HEIGHT
H	WORST BREAKING WAVE HEIGHT
1 AND 2	HYDROGRAPH STEPS

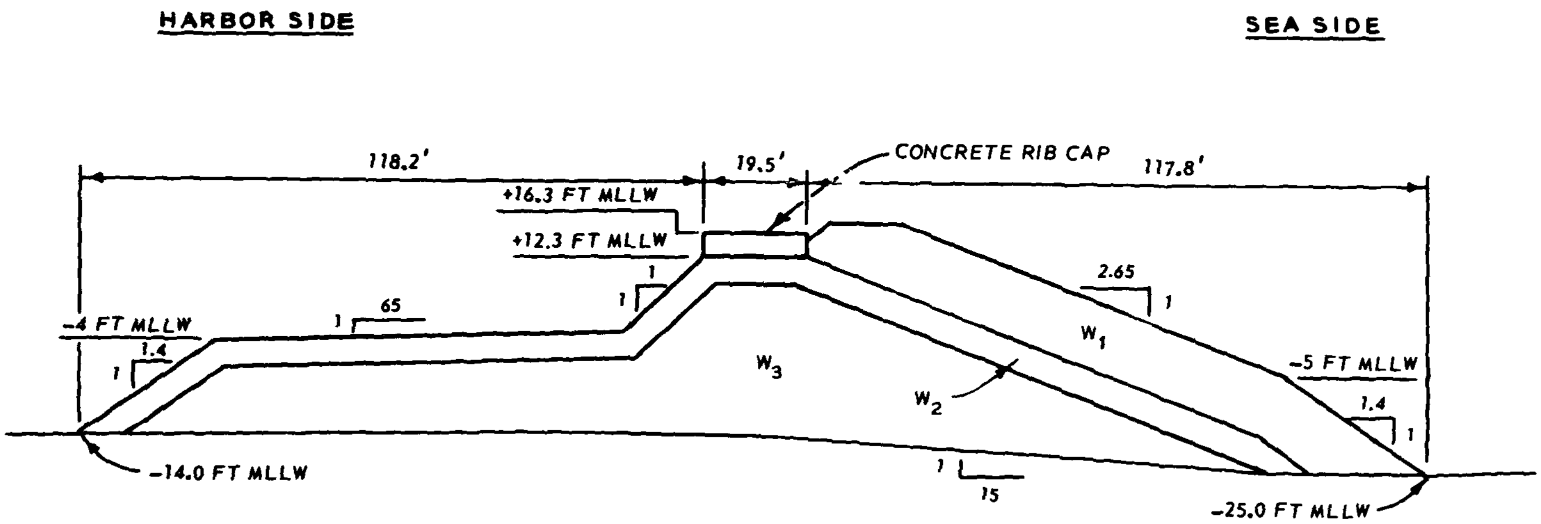
HYDROGRAPH C



LEGEND

T	WAVE PERIOD
H*	SHAKEDOWN WAVE HEIGHT
H	WORST BREAKING WAVE HEIGHT
1 AND 2	HYDROGRAPH STEPS

HYDROGRAPH D

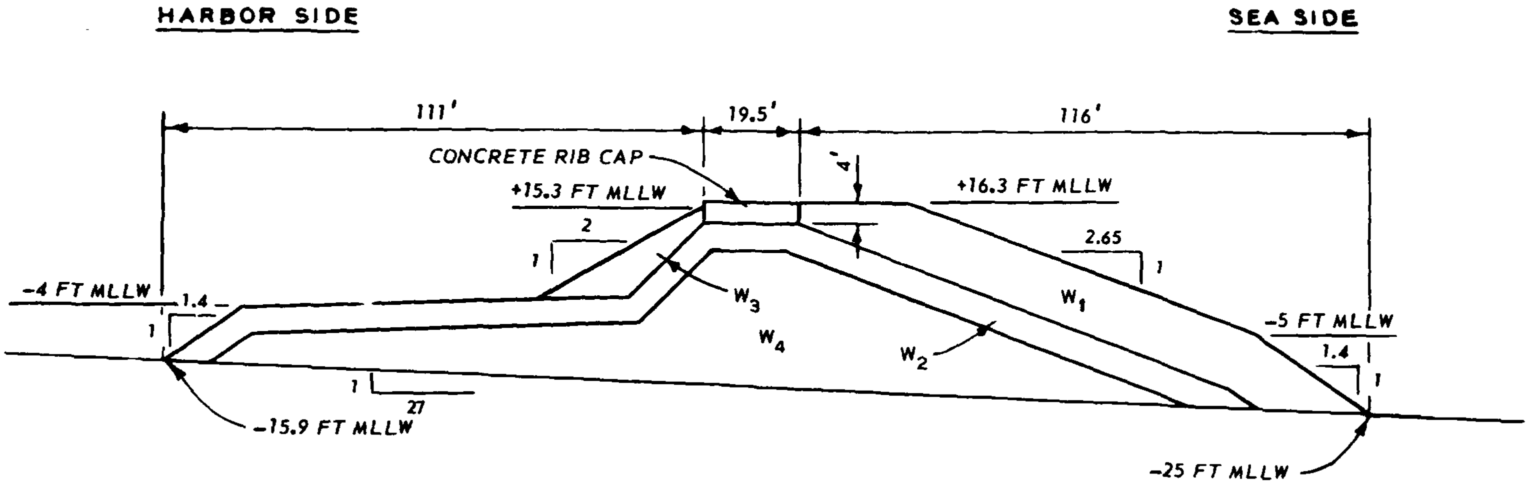


PROTOTYPE MATERIAL CHARACTERISTICS

*W_1 = 38,000-LB TRIBARS @ 146 PCF
**W_2 = 16,000-LB ROCK @ 156 PCF
W_3 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* RANDOM-PLACED, TWO LAYERS.
** RANDOM-PLACED, ONE LAYER.

EXISTING CONDITION
WEST BREAKWATER AT STA 21+25



MATERIAL CHARACTERISTICS

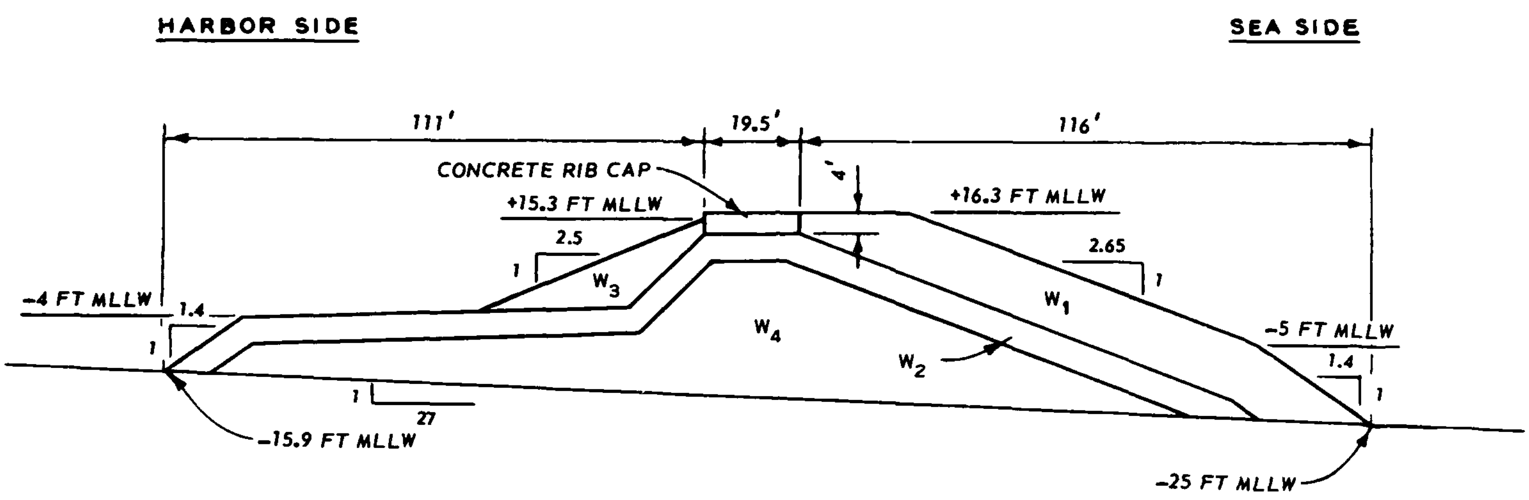
MODEL	PROTOTYPE
W_1 = 0.816-LB TRIBARS @ 141.7 PCF	W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 0.242-LB ROCK @ 165 PCF	W_2 = 16,000-LB ROCK @ 156 PCF
* W_3 = 0.206-LB TO 0.278-LB ROCK @ 165 PCF	W_3 = 13,600-LB TO 18,400-LB ROCK @ 156 PCF
W_4 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF	W_4 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* RANDOM PLACEMENT, VARYING FROM ONE TO THREE LAYERS.

PLAN 1

WEST BREAKWATER AT STA 21+25

SECTION 4



MATERIAL CHARACTERISTICS

MODEL

W_1 = 0.816-LB TRIBARS @ 141.7 PCF
W_2 = 0.242-LB ROCK @ 165 PCF
* W_3 = 0.206-LB TO 0.278-LB ROCK @ 165 PCF
W_4 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF

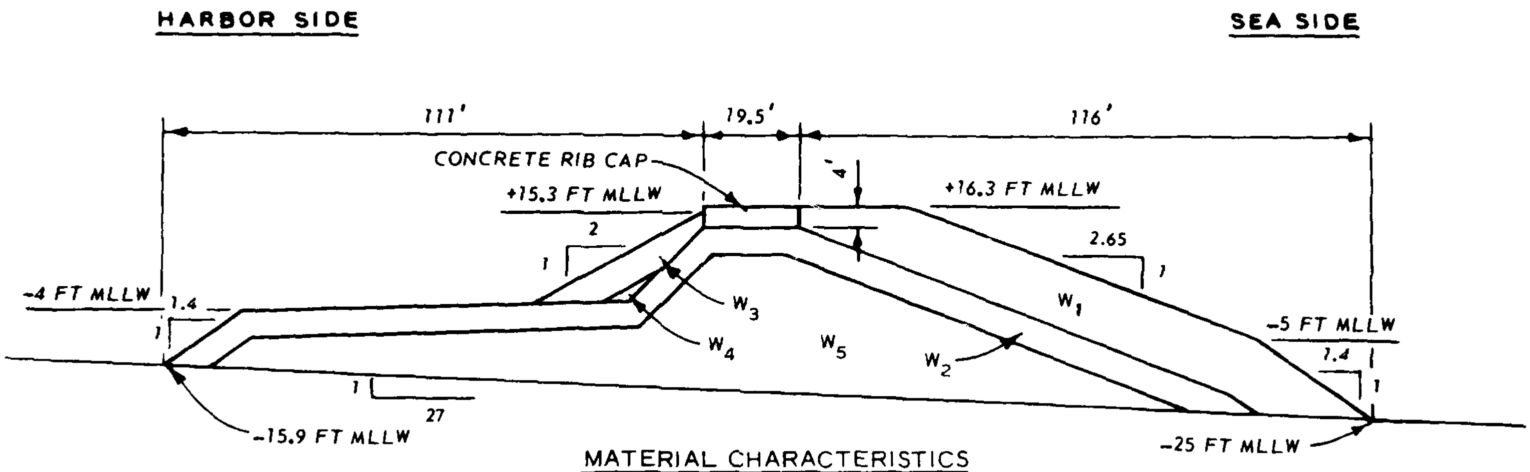
PROTOTYPE

W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 16,000-LB ROCK @ 156 PCF
W_3 = 13,600-LB TO 18,400-LB ROCK @ 156 PCF
W_4 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* RANDOM PLACEMENT, VARYING FROM ONE TO THREE LAYERS.

PLAN 1-1

WEST BREAKWATER AT STA 21+25
SECTION 4-1

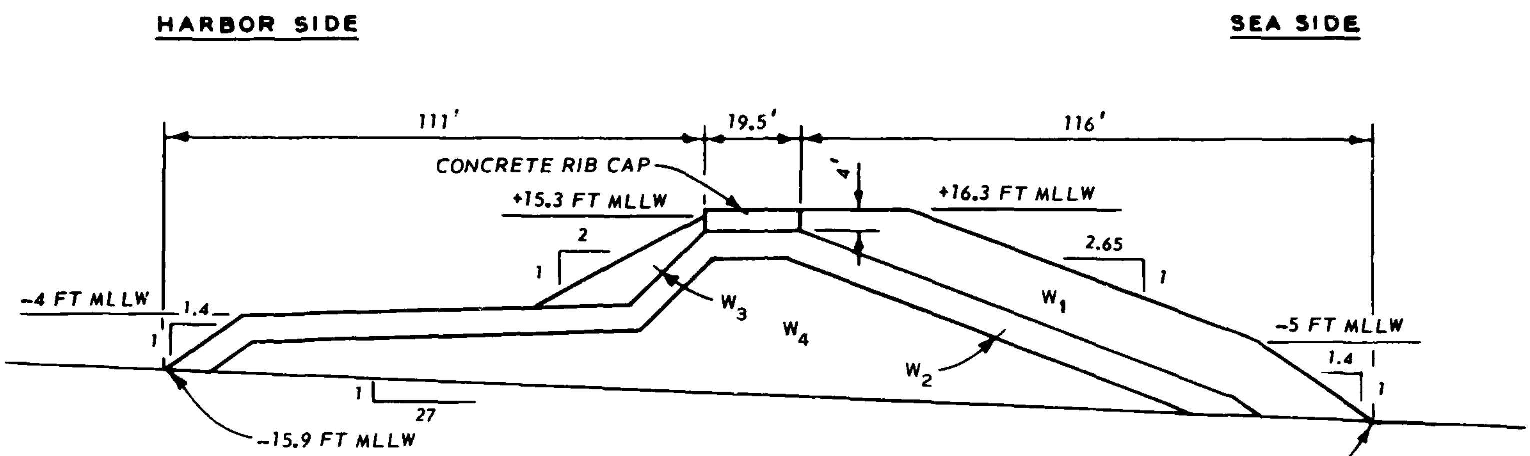


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.816-LB TRIBARS @ 141.7 PCF	W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 0.242-LB ROCK @ 165 PCF	W_2 = 16,000-LB ROCK @ 156 PCF
*W_3 = 0.079-LB DOLOSSE @ 142 PCF	W_3 = 3,740-LB DOLOSSE @ 146 PCF
**W_3 = 0.153-LB DOLOSSE @ 139.8 PCF	W_3 = 6,765-LB DOLOSSE @ 146 PCF
*W_4 = 0.012-LB ROCK @ 165 PCF	W_4 = 800-LB ROCK @ 156 PCF
**W_4 = 0.020-LB ROCK @ 165 PCF	W_4 = 1,350-LB ROCK @ 156 PCF
W_5 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF	W_5 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* PLAN 1A, TWO LAYERS OF RANDOM-PLACED DOLOSSE.
** TWO LAYERS OF DOLOSSE; PLAN 1B, RANDOM PLACEMENT; PLAN 1B-1, RANDOM PLACEMENT ON SLOPE WITH SPECIAL TOE PLACEMENT.

PLANS 1A, 1B, AND 1B-1
WEST BREAKWATER AT STA 21+25
SECTIONS 4A, 4B, AND 4B-1

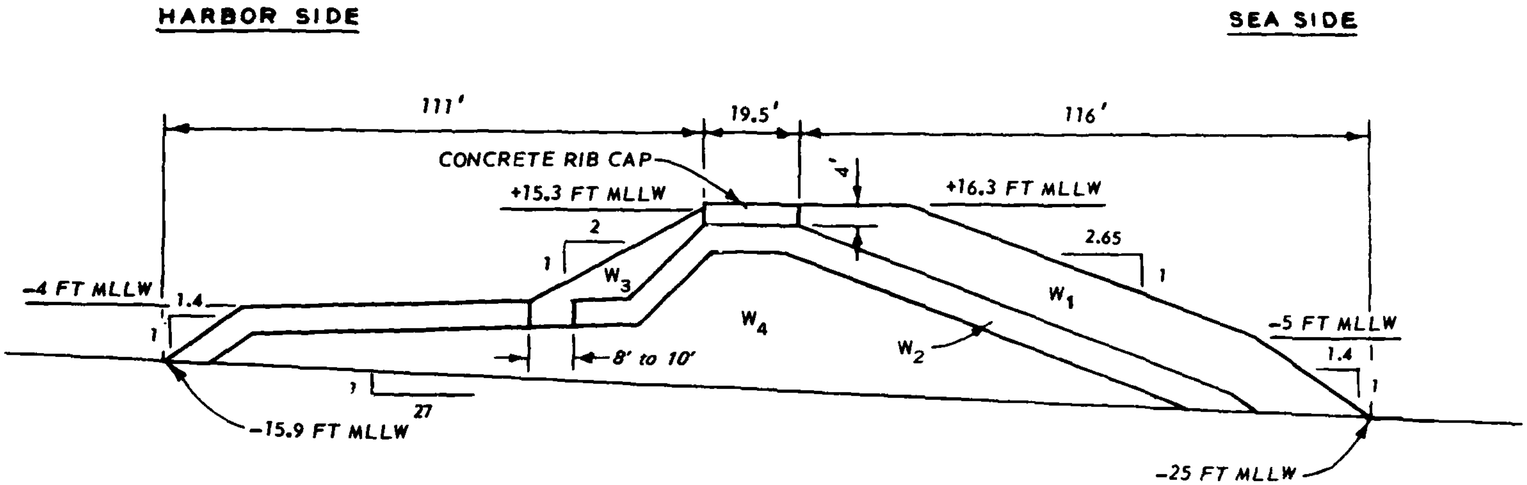


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.816-LB TRIBARS @ 141.7 PCF	W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 0.242-LB ROCK @ 165 PCF	W_2 = 16,000-LB ROCK @ 156 PCF
*W_3 = 0.234-LB DOLOSSE @ 137.7 PCF	W_3 = 9,670-LB DOLOSSE @ 146 PCF
**W_3 = 0.285-LB DOLOSSE @ 142.2 PCF	W_3 = 15,325-LB DOLOSSE @ 146 PCF
W_4 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF	W_4 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* PLAN 1C, TWO LAYERS OF DOLOSSE, RANDOM PLACEMENT ON SLOPE WITH SPECIAL TOE PLACEMENT.
** TWO LAYERS OF DOLOSSE, RANDOM PLACEMENT ON SLOPE; PLAN 1D, SPECIAL TOE PLACEMENT; PLAN 1D-1, EXTRA SPECIAL TOE PLACEMENT.

PLANS 1C, 1D, AND 1D-1
WEST BREAKWATER AT STA 21+25
SECTIONS 4E AND 4F



MATERIAL CHARACTERISTICS

MODEL

W_1 = 0.816-LB TRIBARS @ 141.7 PCF
W_2 = 0.242-LB ROCK @ 165 PCF
*W_3 = 0.285-LB DOLOSSE @ 142.2 PCF
W_4 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF

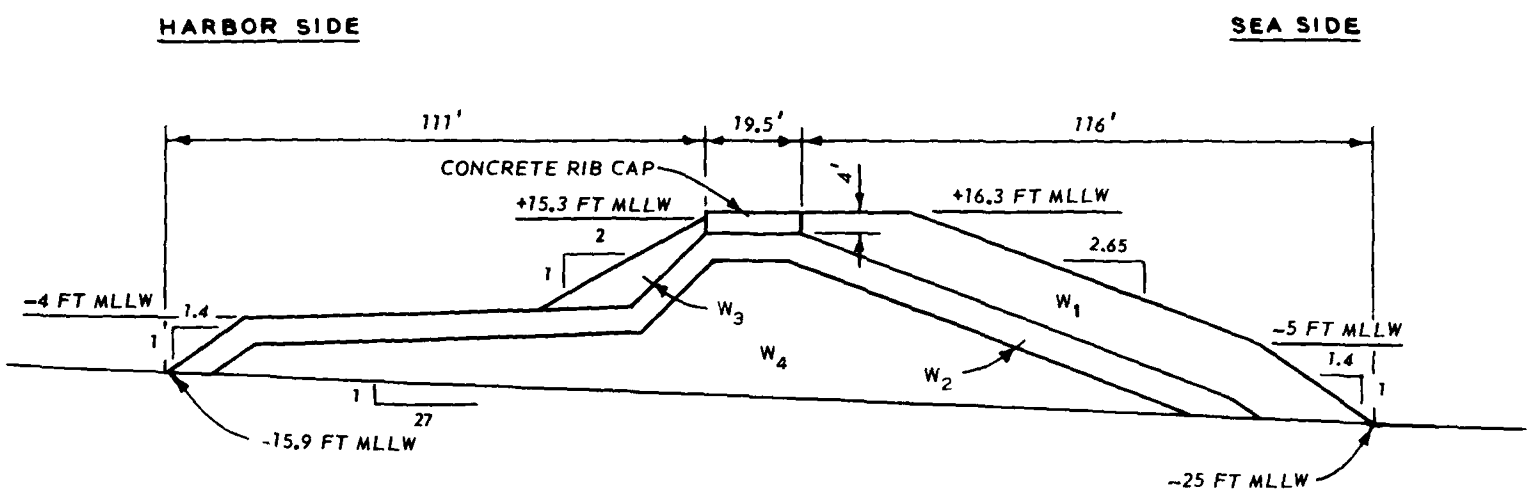
PROTOTYPE

W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 16,000-LB ROCK @ 156 PCF
W_3 = 15,325-LB DOLOSSE @ 146 PCF
W_4 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* TWO LAYERS OF DOLOSSE, RANDOM PLACEMENT ON SLOPE WITH SPECIAL PLACEMENT AND TRENCHING OF TOE UNITS.

PLAN 1D-2

WEST BREAKWATER AT STA 21+25
SECTION 4F-1



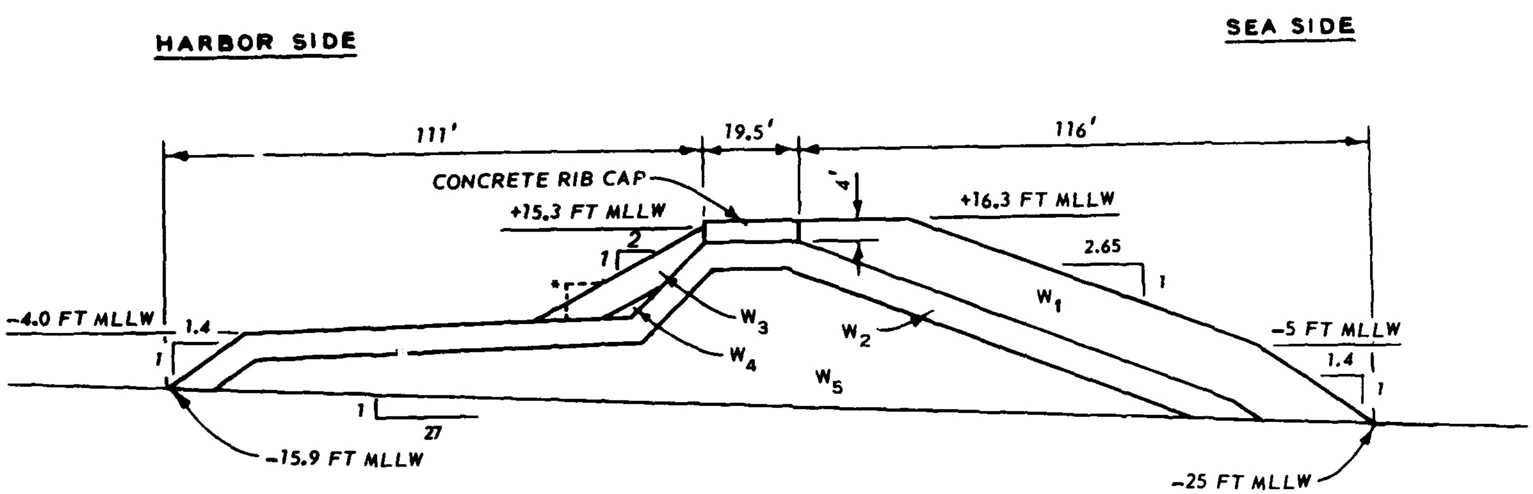
MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.816-LB TRIBARS @ 141.7 PCF	W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 0.242-LB ROCK @ 165 PCF	W_2 = 16,000-LB ROCK @ 156 PCF
*W_3 = 0.442-LB DOLOSSE @ 140.6 PCF	W_3 = 20,945-LB DOLOSSE @ 146 PCF
W_4 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF	W_4 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* TWO LAYERS OF DOLOSSE, RANDOM PLACEMENT ON SLOPE WITH EXTRA SPECIAL TOE PLACEMENT.

PLAN 1E

WEST BREAKWATER AT STA 21+25
SECTION 4G



MATERIAL CHARACTERISTICS

MODEL

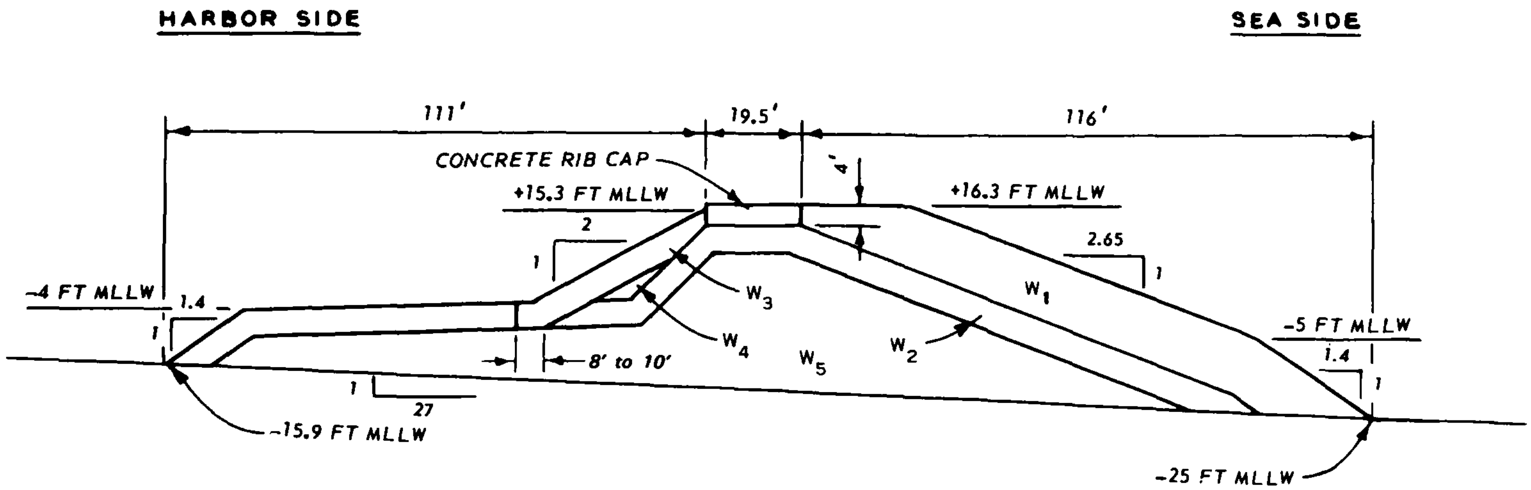
W_1 = 0.816-LB TRIBARS @ 141.7 PCF
W_2 = 0.242-LB ROCK @ 165 PCF
**W_3 = 0.290-LB TRIBARS @ 140.4 PCF
***W_3 = 0.427-LB TRIBARS @ 141.7 PCF
**W_4 = 0.020-LB ROCK @ 165 PCF
***W_4 = 0.030-LB ROCK @ 165 PCF
W_5 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF

PROTOTYPE

W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 16,000-LB ROCK @ 156 PCF
W_3 = 13,065-LB TRIBARS @ 146 PCF
W_3 = 20,000-LB TRIBARS @ 146 PCF
W_4 = 1,307-LB ROCK @ 156 PCF
W_4 = 2,000-LB ROCK @ 156 PCF
W_5 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* TOE GEOMETRY FOR PLAN 1G AND FOR SECOND TEST OF PLAN 1F.
** PLAN 1F, ONE LAYER OF UNIFORM-PLACED TRIBARS.
*** PLAN 1G, ONE LAYER OF UNIFORM-PLACED TRIBARS.

PLANS IF AND IG
WEST BREAKWATER AT STA 21+25
SECTIONS 4C AND 4D



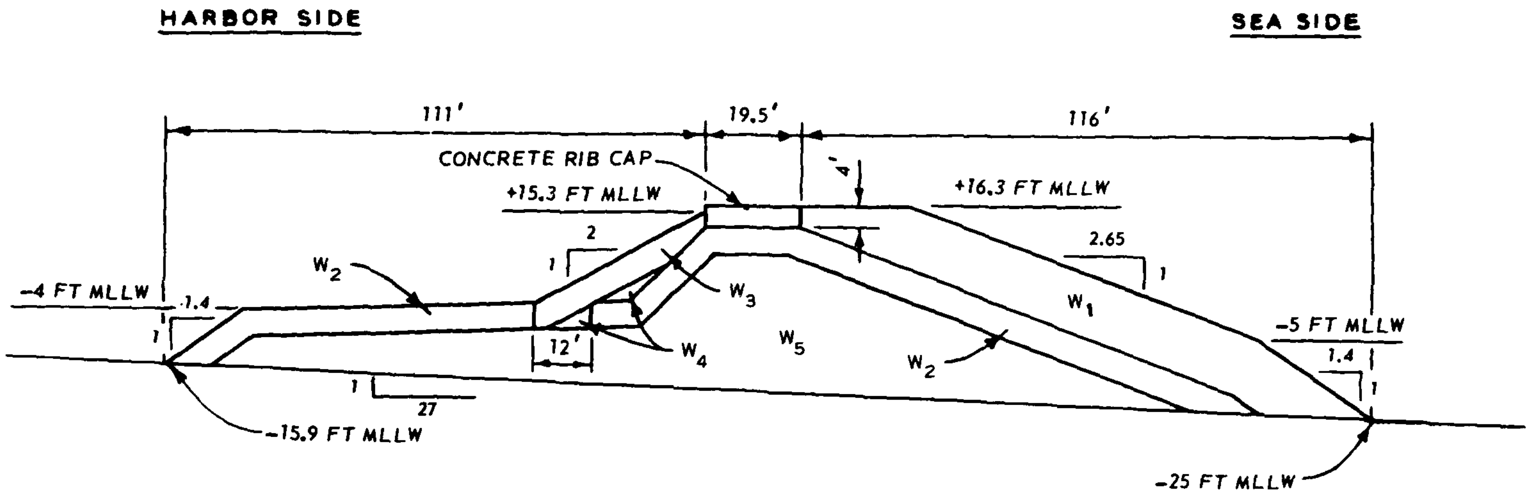
MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.816-LB TRIBARS @ 141.7 PCF	W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 0.242-LB ROCK @ 165 PCF	W_2 = 16,000-LB ROCK @ 156 PCF
* W_3 = 0.290-LB TRIBARS @ 140.4 PCF	W_3 = 13,065-LB TRIBARS @ 146 PCF
W_4 = 0.020-LB ROCK @ 165 PCF	W_4 = 1,307-LB ROCK @ 156 PCF
W_5 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF	W_5 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* ONE LAYER OF UNIFORM-PLACED TRIBARS WITH TOE UNITS TRENCHED.

PLAN 1F-1

WEST BREAKWATER AT STA 21+25
SECTION 4C-1



MATERIAL CHARACTERISTICS

MODEL

W_1 = 0.816-LB TRIBARS @ 141.7 PCF
W_2 = 0.242-LB ROCK @ 165 PCF
*W_3 = 0.427-LB TRIBARS @ 141.7 PCF
W_4 = 0.030-LB ROCK @ 165 PCF
W_5 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF

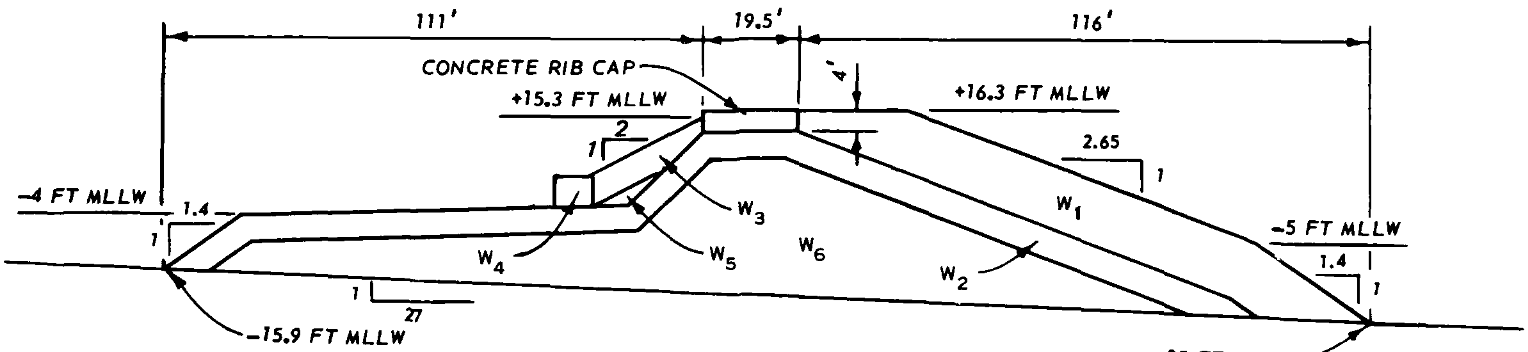
PROTOTYPE

W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 16,000-LB ROCK @ 156 PCF
W_3 = 20,000-LB TRIBARS @ 146 PCF
W_4 = 2,000-LB ROCK @ 156 PCF
W_5 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* ONE LAYER OF UNIFORM-PLACED TRIBARS WITH TOE TRENCHED.

PLAN 1G-1

WEST BREAKWATER AT STA 21+25
SECTION 4D-1



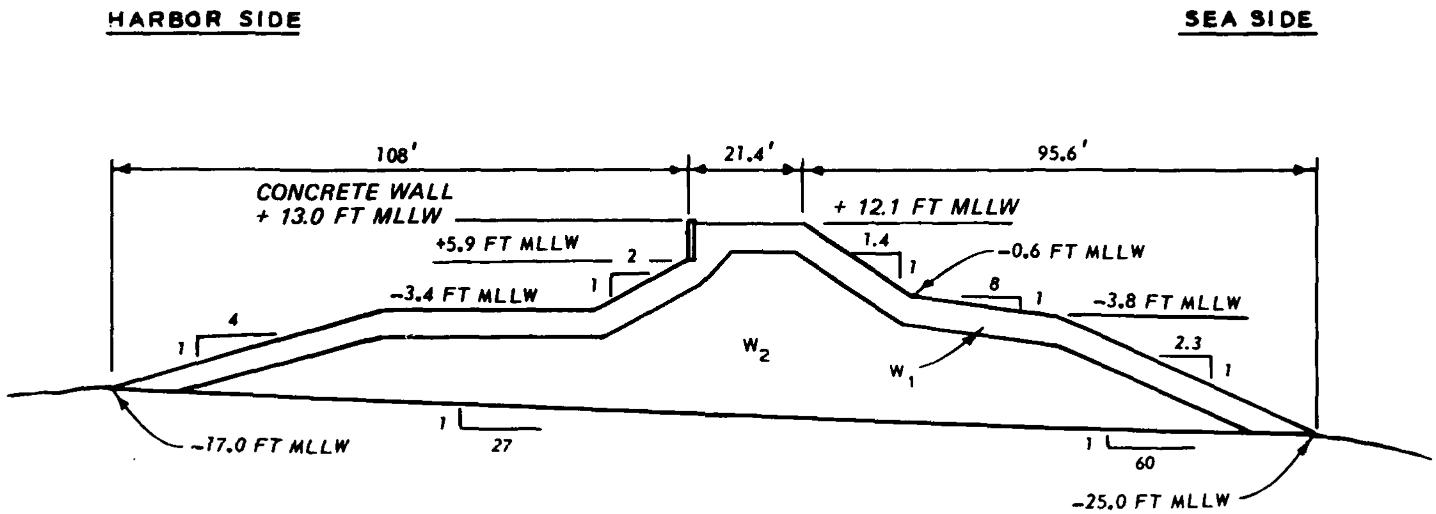
MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.816-LB TRIBARS @ 141.7 PCF	W_1 = 38,220-LB TRIBARS @ 146 PCF
W_2 = 0.242-LB ROCK @ 165 PCF	W_2 = 16,000-LB ROCK @ 156 PCF
* W_3 = 0.427-LB TRIBARS @ 141.7 PCF	W_3 = 20,000-LB TRIBARS @ 146 PCF
** W_4 = 0.827-LB TRIBARS @ 140.4 PCF	W_4 = 28,250-LB TRIBARS @ 146 PCF
*** W_4 = 0.816-LB TRIBARS @ 141.7 PCF	W_4 = 38,220-LB TRIBARS @ 146 PCF
W_5 = 0.030-LB ROCK @ 165 PCF	W_5 = 2,000-LB ROCK @ 156 PCF
W_6 = 0.061-LB TO 0.121-LB ROCK @ 165 PCF	W_6 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* ONE LAYER OF UNIFORM-PLACED TRIBARS.
** PLAN 1H, SINGLE ROW OF TRIBARS (ONE LAYER, UNIFORM PLACEMENT).
*** PLAN 1I, SINGLE ROW OF TRIBARS (ONE LAYER, UNIFORM PLACEMENT).

PLANS 1H AND 1I

WEST BREAKWATER AT STA 21+25
SECTIONS 4H AND 4I

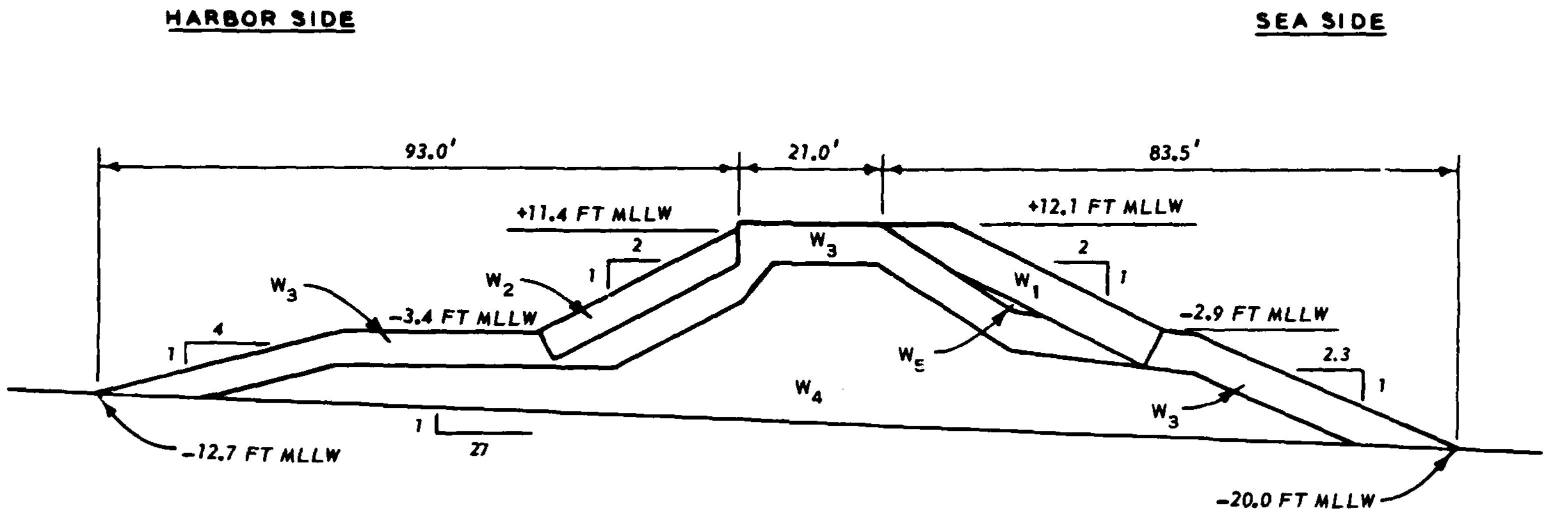


PROTOTYPE MATERIAL CHARACTERISTICS

*W_1 = 16,000-LB ROCK @ 156 PCF
W_2 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* RANDOM-PLACED, ONE LAYER.

EXISTING CONDITION
WEST BREAKWATER AT STA 18+50



MATERIAL CHARACTERISTICS

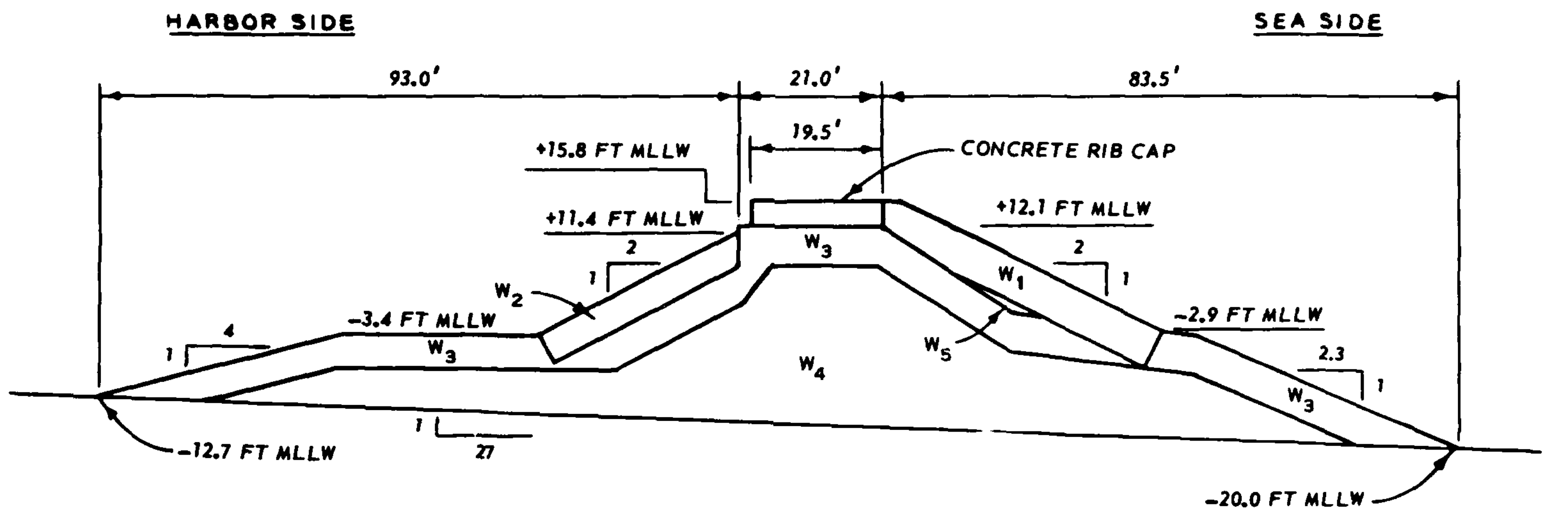
MODEL	PROTOTYPE
*W_1 = 0.827-LB TRIBARS @ 140.4 PCF	W_1 = 21,758-LB TRIBARS @ 146 PCF
*W_2 = 0.290-LB TRIBARS @ 140.4 PCF	W_2 = 10,064-LB TRIBARS @ 146 PCF
W_3 = 0.315-LB ROCK @ 165 PCF	W_3 = 16,000-LB ROCK @ 156 PCF
W_4 = 0.079-LB TO 0.157-LB ROCK @ 165 PCF	W_4 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF
W_5 = 0.043-LB ROCK @ 165 PCF	W_5 = 2,176-LB ROCK @ 156 PCF

* ONE LAYER OF UNIFORM-PLACED TRIBARS WITH THE TOE TRENCHED.

PLAN 2

WEST BREAKWATER AT STA 18+50

SECTION 3

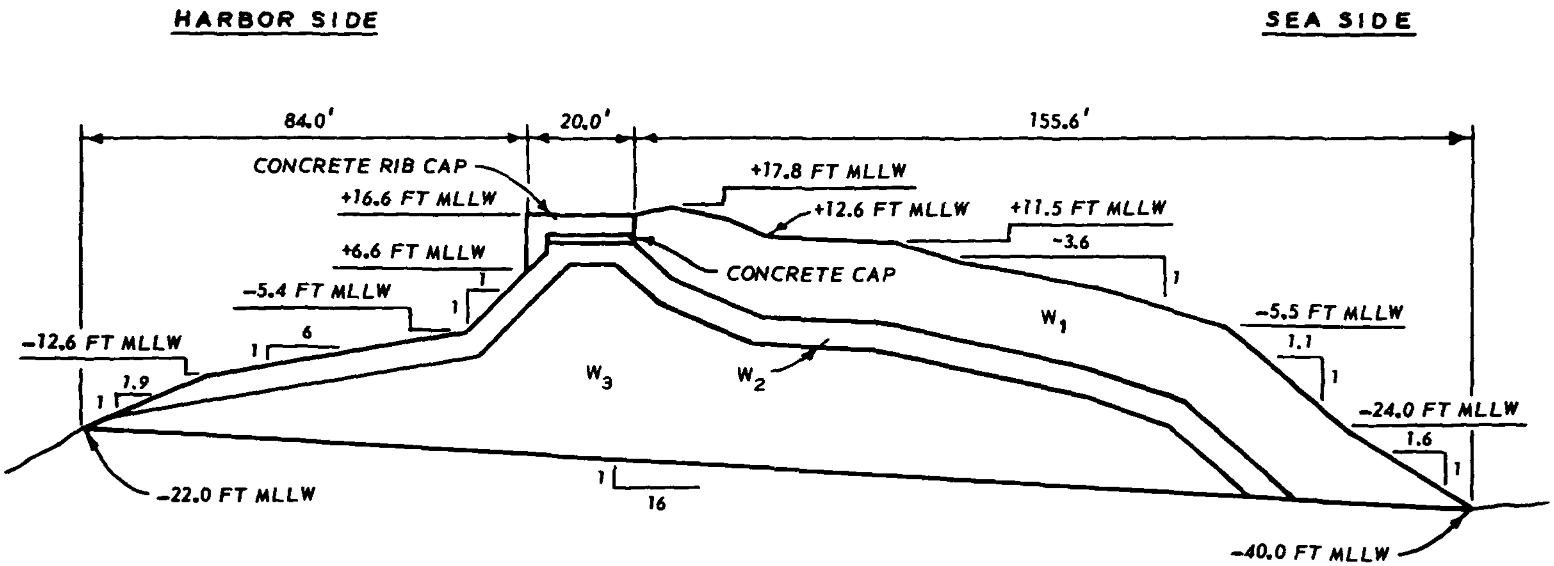


MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
* W_1 = 0.827-LB TRIBARS @ 140.4 PCF	W_1 = 21,758-LB TRIBARS @ 146 PCF
* W_2 = 0.290-LB TRIBARS @ 140.4 PCF	W_2 = 10,064-LB TRIBARS @ 146 PCF
W_3 = 0.315-LB ROCK @ 165 PCF	W_3 = 16,000-LB ROCK @ 156 PCF
W_4 = 0.079-LB TO 0.157-LB ROCK @ 165 PCF	W_4 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF
W_5 = 0.043-LB ROCK @ 165	W_5 = 2,176-LB ROCK @ 156 PCF

* ONE LAYER OF UNIFORM-PLACED TRIBARS WITH THE TOE TRENCHED.

PLAN 2A
WEST BREAKWATER AT STA 18 + 50
SECTION 3A

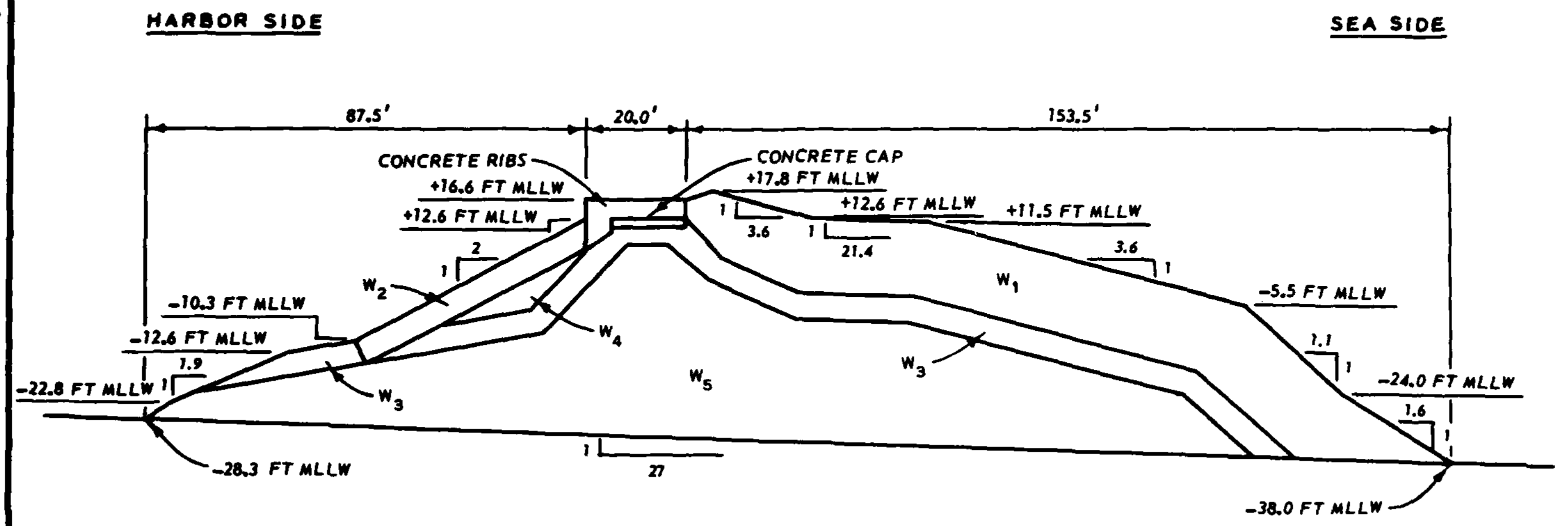


PROTOTYPE MATERIAL CHARACTERISTICS

*W_1 = 70,000-LB TRIBARS @ 146 PCF
**W_2 = 16,000-LB ROCK @ 156 PCF
W_3 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* RANDOM-PLACED, TWO LAYERS.
** RANDOM-PLACED, ONE LAYER.

EXISTING CONDITION
EAST BREAKWATER AT STA 26+10



MATERIAL CHARACTERISTICS

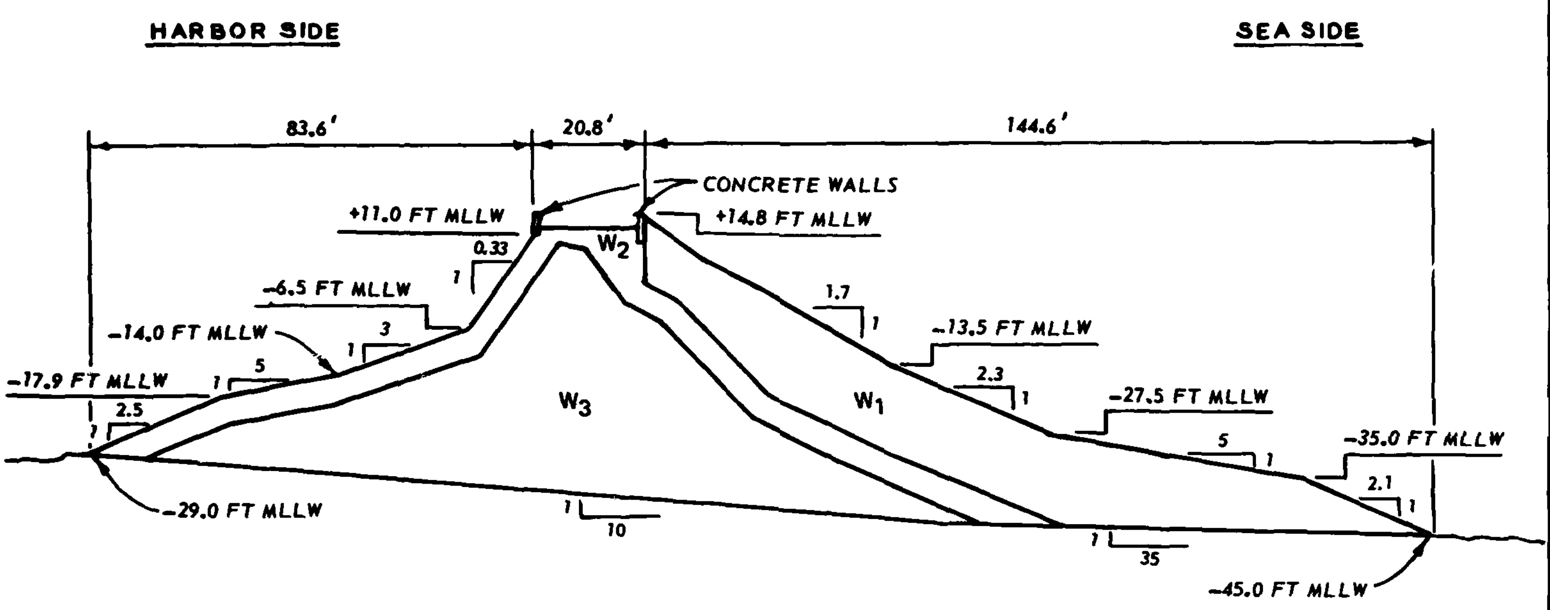
MODEL	PROTOTYPE
W_1 = 1.125-LB TRIBARS @ 141.0 PCF	W_1 = 70,800-LB TRIBARS @ 146 PCF
*W_2 = 0.290-LB TRIBARS @ 140.4 PCF	W_2 = 17,920-LB TRIBARS @ 146 PCF
W_3 = 0.177-LB ROCK @ 165 PCF	W_3 = 16,000-LB ROCK @ 156 PCF
W_4 = 0.020-LB ROCK @ 165 PCF	W_4 = 1,790-LB ROCK @ 156 PCF
W_5 = 0.044-LB TO 0.088-LB ROCK @ 165 PCF	W_5 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* ONE LAYER OF UNIFORM-PLACED TRIBARS WITH TOE TRENCHED.

PLAN 3

EAST BREAKWATER AT STA 28+10

SECTION 2

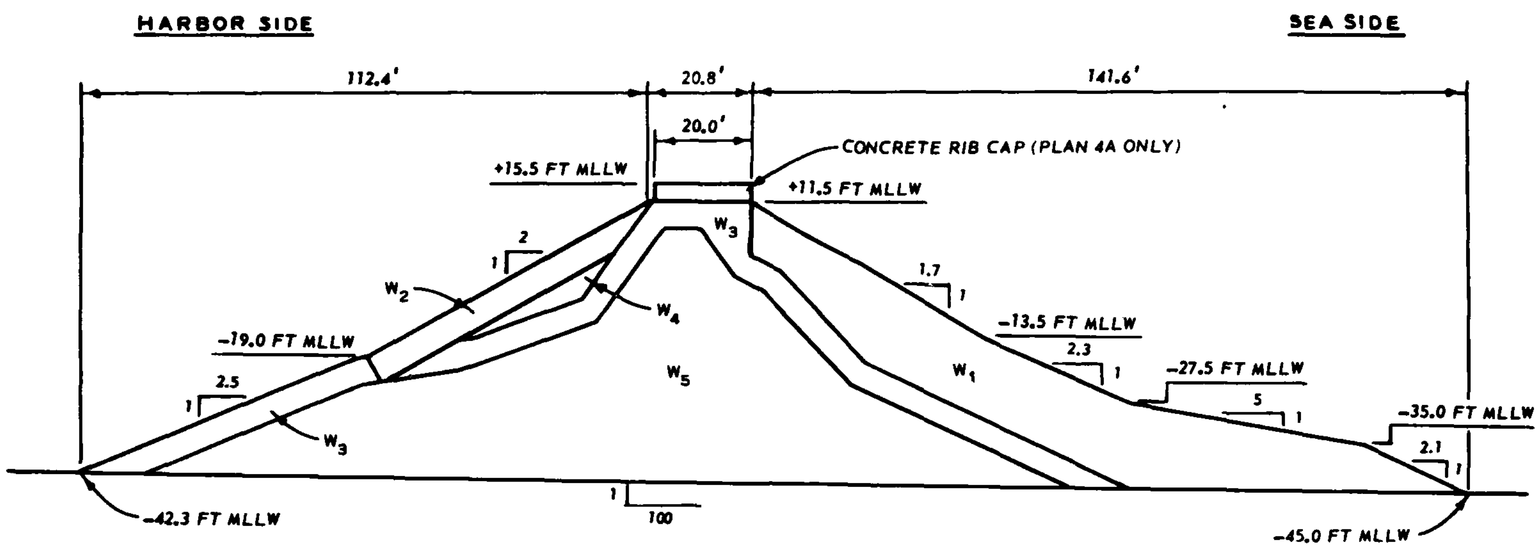


PROTOTYPE MATERIAL CHARACTERISTICS

* W_1 = 60,000-LB DOLOSSE @ 146 PCF
** W_2 = 16,000-LB ROCK @ 156 PCF
W_3 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* RANDOM-PLACED, TWO LAYERS.
** RANDOM-PLACED, ONE LAYER.

EXISTING CONDITION
EAST BREAKWATER AT STA 23+35



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.882-LB DOLOSSE @ 142.7 PCF	W_1 = 58,500-LB DOLOSSE @ 146 PCF
*W_2 = 0.290-LB TRIBARS @ 140.4 PCF	W_2 = 17,920-LB TRIBARS @ 146 PCF
W_3 = 0.177-LB ROCK @ 165 PCF	W_3 = 16,000-LB ROCK @ 156 PCF
W_4 = 0.020-LB ROCK @ 165 PCF	W_4 = 1,790-LB ROCK @ 156 PCF
W_5 = 0.044-LB TO 0.088-LB ROCK @ 165 PCF	W_5 = 4,000-LB TO 8,000-LB ROCK @ 156 PCF

* ONE LAYER OF UNIFORM-PLACED TRIBARS WITH TOE TRENCHED.

PLANS 4 AND 4A
EAST BREAKWATER AT STA 23 + 35
SECTIONS 1 AND 1A

APPENDIX A: NOTATION

A	Area, ft^2
H	Wave height, ft
L	Length, linear scale, ft
mllw	Mean lower low water
S	Specific gravity
sta	Station, surveying location where observations are taken
swl	Still-water level
T	Wave period, time, sec
V	Volume, ft^3
W	Weight of individual stone or armor unit, lb
q	Specific weight, pcf

Subscripts

a	Refers to the ratio of model to prototype quantities
m	Refers to model quantities
p	Refers to prototype quantities
r	Refers to armor stone or armor units
w	Refers to water in which structure is situated
1-5	Refers to different sizes and types of construction material

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Markle, Dennis G.
Kahului Breakwater Stability Study Kahului, Maui, Hawaii : Hydraulic Model Investigation / by Dennis G. Markle (Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. : available from NTIS, 1982.
132 p. in various pagings, 22 p. of plates ; ill. ; 27 cm. -- (Technical report ; HL-82-14)
Cover title.
"July 1982."
Final report.
"Prepared for U.S. Army Engineer Division, Pacific Ocean."

1. Breakwaters. 2. Harbors--Hawaii. 3. Hydraulic models. 4. Kahului Harbor (Hawaii). I. United States. Army. Corps of Engineers. Pacific Ocean Division. II. U.S. Army Engineer Waterways Experiment Station. Hydraulics Division. III. Title IV. Series:

Markle, Dennis G.
Kahului Breakwater Stability Study Kahului : ... 1982.
(Card 2)

Technical report (U.S. Army Engineer Waterways Experiment Station) ; HL-82-14.
TA7.W34 no.HL-82-14

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